RK&K

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MEMORANDUM

Date: April 26, 2016

To: **Delaware Department of Transportation**
800 Bay Road
Dover, Delaware 19903

From: Eric M. Klein, P.E., D.GE EMK
Bibek B. Shrestha, P.E BBS

CC: Nancy R. Bergeron, P.E.

Re: Additional Foundation Recommendations
Christina River Bridge
City of Wilmington, Delaware
Comm. No. 04-130-03

This memorandum is a supplement to the Final Foundation Report (FFR), dated January 29, 2016 for the Christina River Bridge in the City of Wilmington, Delaware. This memorandum provides the following analysis and recommendations for the proposed construction:

- West Approach Ramp - Settlement Analysis of Grade Beam supporting the EPS facing panels
- East Approach Ramp - Design of Load Transfer Platform (LTP) for Deep Mixing Method (DMM)
- Roadway Embankment (East of STA 444+00) - Lateral squeeze analysis
- Special Consideration – Seismic monitoring

The following recommendations have been developed on the basis of the project characteristic and subsurface conditions described in the Final Foundation Report dated January 29, 2016. If there are any significant changes to the project characteristics or if significantly different subsurface conditions are encountered during construction, RK&K should be consulted so that the recommendations of this memorandum can be reviewed and modified as required.

West Approach Ramp

In the FFR, we recommended the west approach ramp west of the underpass be constructed of Expanded Poly-Styrene (EPS). To eliminate any additional settlement a fully compensated embankment should be constructed by undercutting the foundation soils to a depth of 4.5-ft below the existing ground surface. The leveling pad for the proposed EPS retaining wall facing panels will be located at approximately EL +6. The proposed leveling pad for the facing panels will be 2.5-ft wide. We estimate a maximum load increase of approximately 715-psf along the facing panel.

We estimated the total settlement below the facing panel for the EPS retaining wall. The settlement analysis was performed using the geotechnical software application FoSSA 2.0. A maximum total settlement of 0.6-inches, approximately 0.25-inches immediate settlement and 0.35-inches consolidation settlement, is anticipated below the facing panel. The immediate settlement below the leveling pad was also estimated using the semi-empirical strain influence factor proposed by Schmertmann and Hartman (1978). The anticipated immediate settlement using this method is approximately 0.2-inches.

East Approach Ramp

In the FFR, we recommend the east side ramp be construction of MSE wall supported on DMM columns. The soil improvement using DMM will extend from Abutment B at approximately STA 440+70 to STA 443+50. The load from the approach embankment will be transferred to the DMM columns through a geosynthetic reinforced Graded Aggregate Load Transfer Platform (LTP). The LTP will be constructed immediately above the columns to help transfer the load and prevent a "bearing capacity" type of failure above the columns. The LTP also reduces differential settlement for lower height embankments.

The design of the LTP was performed using the Beam (Collin) Method (Ref: *Geosynthetic-Reinforced Column-Support Embankment Design Guidelines* by Collins, Han, and Huang). The thickness of the LTP should be at least one half the clear span between the DMM columns. The vertical load from the soil within the arch and any surcharge load, if the thickness of the embankment is not great enough to develop the full arch, is carried by the reinforcement. The tensile load in the reinforcement was estimated based on tension membrane theory and is a function of the amount of strain in the reinforcement. A minimum of three layers of geosynthetic reinforcement should be installed in the load transfer platform and the initial strain in the geogrid reinforcement should be limited to 5%. The preliminary design of the reinforcement for a clear span between DMM columns of 5-ft and 8-ft are summarized in Table 1.

Table 1 – Summary of Load Transfer Platform Reinforcement

Clear Spacing between DMM columns (ft)	LTP Thickness (ft)	Maximum Design Tensile Load at 5% Strain (lb/ft)	Geogrid Reinforcement	Ultimate Tensile Strength (lb/ft)	Long Term Allowable Design Strength (lb/ft)
5-ft	3-ft	154	Biaxial BX1100	850	237
8-ft	4-ft	386	Biaxial BX1500	1850	516

Roadway Embankment (East of STA 444+00)

The roadway embankment east of STA 444+00 will be constructed using Type F – Common Borrow. The maximum height of embankment fill will be approximately 7-ft above the existing ground surface at approximately STA 448+50. The side slopes of the roadway embankment will be approximately 3(H):1(V). To minimize the effect of the long term settlement we recommend the roadway embankment be constructed with a 2-ft additional surcharge above the proposed grade and quarantined for a minimum time period of 5-months.

We evaluated the potential for lateral squeeze due to the construction of the roadway embankment based on the method proposed by Silvestri, 1983.

$$FS_{Squeezing} = \frac{2c_u}{\gamma D_s \tan\theta} + \frac{4.14\, c_u}{H\gamma} \geq 1.3$$

where,

θ = Angle of Slope
γ = Unit Weight of soil in slope
D_s = Depth of soil beneath slope base of embankment
H = Height of slope
c_u = Undrained shear strength of soft soil beneath slope

The lateral squeeze analysis was performed for the 7-ft fill embankment and for the temporary condition with a 2-ft additional surcharge above the proposed grade. Table 2 summarizes the Factor of Safety for lateral squeezing for the proposed embankment and the additional 2-ft surcharge.

Table 2 – Summary of Factor of Safety for Lateral Squeezing

Location	Embankment	Maximum Height	Factor of Safety
STA 448+50	Proposed Grade	7.0-ft	1.55
	2-ft Additional Surcharge	9.0-ft	1.34

The FS with the 2-ft surcharge is getting close to the minimum recommended. Inclinometers will be installed; if bulging is noted in the inclinometers, a portion of the fill can be removed until the pore pressures dissipate.

Seismic Monitoring

Seismic monitoring is required for all construction operations within a distance of 50-feet of the existing utilities that have the potential to produce vibrations at damaging levels, such as pile driving or significant truck traffic. The vibrations should be monitored for structures and utilities within 50-ft of any drilled shaft installation or sheet pile installation. Drilled shaft drilling will cause only about 0.089-in/sec at a distance of 25-ft, so this will not likely cause any significant disturbances. However, installing the casing may cause between 0.17 to 0.734-in/sec at 25-ft depending on the method of installing the casing, so we recommend vibration monitoring for casing and pile driving installation.

A firm specializing in this vibration monitoring should be retained by the Contractor to monitor the construction induced vibrations. A detailed vibration monitoring plan should be submitted for the Engineer's approval. The plan would include the monitoring locations, the type of equipment to be used, qualifications of the monitoring personnel, and requirements for the timely presentation of monitoring data to the Engineer.

Vibrations can also effect green concrete. The Contractor should monitor construction activities adjacent to freshly placed concrete utilizing one of the two seismic monitoring options listed below. Construction activities to be limited during this period include, but not limited to, drilled shaft installation and use of any type of heavy construction equipment.

Option 1:

Where vibration monitoring around freshly poured concrete is performed the following Peak Particle Velocity (PPV) limits and distances shall be maintained:

Peak Particle Velocities for Concrete at Different Ages.

Age of concrete at which vibration occurs	Permitted peak particle velocity, (in/sec)
0-10 hrs	0.2
10-24 hrs	0.4
More than 24-hrs	2.0

Clear Distance between construction operations and freshly poured concrete at different ages.

Age of concrete	Clear Distance (ft)
0-2 days	50
2-14 days	20

The vibration shall be monitored continuously with a real time data acquisition system with an alarm system to notify the Contractor if vibration exceeds the limiting values.

Option 2:

Where vibration is not monitored the following limits shall be maintained:

Clear distance between construction operations and freshly poured concrete at different ages

Age of concrete	Clear Distance (ft)
0-2 days	100
2-14 days	20

Peak particle velocity (PPV) at existing adjacent structures or utilities shall not exceed that shown in the Office of Surface Mining (OSM) Method 3 Figure shown in Figure 1. The criteria for drywall should be used for all structures and utilities except those that actually are constructed of plaster or otherwise noted. These limits may be adjusted by the Engineer based on any evidence of damage to structures.

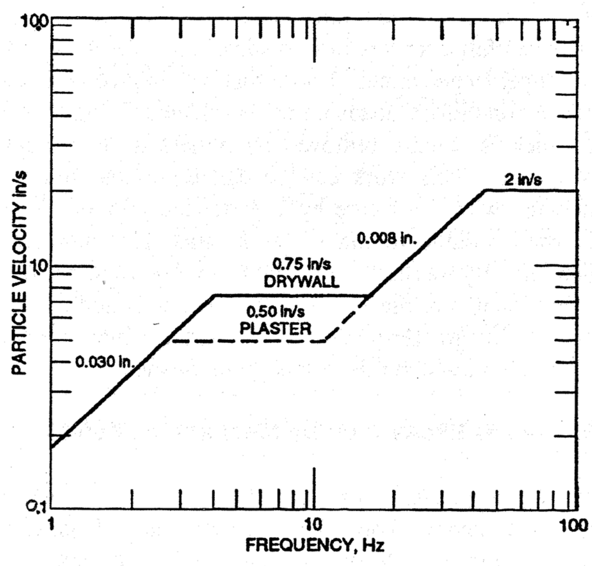

Figure 1 – OSM Method 3

Attachments

Calculations

- Settlement Analysis of Grade Beam Supporting EPS Facing Panels
- Design of Load Transfer Platform for Deep Mixing Method
- Roadway Embankment STA 447+00 to STA 450+00 Lateral Squeeze Analysis

M:\projects\2004\04130_crb\Geotech\GER\2016 Supplement Memo\Geotechnical Memo 04-27-16.docx

RK&K
81 Mosher Street
Baltimore, Maryland 21217

GEOTECHNICAL QUALITY CONTROL SIGN OFF SHEET

Project Name: Christina River Bridge

Commission Number: 04-130-03G

Client Name:

Document Title: Settlement Analysis of Grade Beam Supporting EPS Facing Panels

Document Type: (Check One)
- ☐ Proposal
- ☐ Report/Memo
- ☒ Calculations
- ☐ Boring Logs
- ☐ Specification
- ☐ Other: ____

Project Milestone: ____ **Due Date:** ____

This is to certify that I was the originator of this work product, and that I have carefully checked my work prior to passing it on for Quality Control checking.

Printed Name: Arjun Roy

Signature: [signature] **Date:** 03/24/2016

	Print Name	Signature	Date
☑ Checked:	ERIC M KLEIN	[signature]	4/20/16
☑ Corrected:	BIBEK SHRESTHA	Bibek Shrestha	4/21/16
☑ Checked:	ERIC M KLEIN	[signature]	4/26/16
☐ Corrected:			
☐ Checked:			
☐ Project Engr:			
☑ PM/Manager:	ERIC M KLEIN	[signature]	4/26/16

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GEOTECHNICAL CALCULATION QUALITY CONTROL CHECKLIST

Project: Christina River Bridge **Comm. No:** 04-130-03G

Title of Calculation Set: Settlement Analysis of Grade Beam Supporting EPS Facing Panels

RK&K Project Manager: ______ **Coordinating Discipline POC:** ______

Calculations Prepared By: Arjun Roy **Design Method/Code:** FoSSA 2.0

File Path on Network: M:\projects\2004\04130_crb\Geotech\Calcs\Final Design Calcs 2016\Grade Beam

Subsurface Characterization Approved By: ______

Initial Calculations: Signature of Originator: Arjun Date: 03/24/2016

- ☒ Geotech Quality Control Sign Off Sheet is provided.
- ☒ Calculations are neat, legible and understandable.
- ☒ Variable inputs are highlighted and check confirmations are clearly marked in calculation spreadsheets.
- ☒ Table of Contents is provided.
- ☒ Heading on each page filled out, including computer input and output.
- ☒ The first page contains purpose of the calculation.
- ☒ References for design criteria, data, methods, formulas and computer program names and version numbers are given, and included in appendices as necessary.
- ☒ All assumptions are listed, explained, and those needed to be verified later are flagged.
- ☒ All input by others is attached and includes originator, date provided. (i.e. Structures loads, concrete/steel properties, etc.)
- ☒ Can the analytical steps involved be followed independent of the originator?
- ☒ Do the calculations agree with the other inter-department / project documents?
- ☒ Footer with file path/tab name and print date is provided on calculation spreadsheets.
- ☒ Detailed CAD ready sketches are provided that clearly depict the design requirements.
- ☒ Calculation Summary with recommendations included in front of calculation set.
- ☒ All calculations are checked and each calculation sheet is initialed by the originator before giving to the checker.

Finalized Calculations: Signature of Originator: ______ Date: ______

- ☐ All pages are numbered sequentially and cross-references are complete and accurate.
- ☐ All calculations are checked and each calculation sheet initialed by the checker.
- ☐ Calculation set reviewed by Project Manager.
- ☐ Has this calculation been superseded? Reference new calculation, if applicable.

RK&K

Subject Settlement of Grade Beam Supporting EPS Fill Page TOC of

Christina River Bridge Cm. No. 04150-0369

Prepared By ACR Date 03/24/16 Checked By Date

Table of Content

Purpose: Settlement Analysis of Grade Beam Supporting EPS Facing Panel.

Assumptions:

▷ The Grade Beam will be founded on Existing fill over soft stratum Ia.

References:

1) FoSSA 2.0

Design Load:

service pressure = 715 psf

Summary of settlement Analysis:

Elastic settlement (up to 10 ft deep) = 0.24 in. (pg 11)

Consolidation settlement (layer 2) = 0.36 in (pg 12)

ELASTIC SETTLEMENT (SCHMERTMAN METHOD) = 0.17 in (pg 15)

RE: RW1 and RW2 Levelling Pad Bearing Pressure

Kimberly Duong

Fri 3/11/2016 3:30 PM

To Bibek Shrestha <bshrestha@rkk.com>;

Looks like service pressure is about 715 psf.

KIMBERLY M. DUONG, PE
Project Engineer, Structures

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RESPONSIVE PEOPLE | CREATIVE SOLUTIONS

From: Bibek Shrestha
Sent: Friday, March 11, 2016 3:27 PM
To: Kimberly Duong <kduong@rkk.com>
Subject: Re: RW1 and RW2 Levelling Pad Bearing Pressure

Kim,

Does he have a service load because I will need that to run my settlement.

Thank you,

Bibek

Bibek B. Shrestha, P.E.
Project Engineer, Geotechnical

C_c $CC = \frac{C_c}{1+e_o}$

Christina River Bridge

West Bank (Strata Ia)

Boring No.	Depth	Elevation	Overburden	P_c	OCR	C_c	C_r	e_o	Cv	CC	CR	C_α	C_α'	
AA-1	21	-17.2	0.35	0.90	2.57	0.5	0.08	1.904	0.09	0.1722	0.0275	0.017	0.0059	0.0340
AA-2	12	-7.7	0.72	0.50	0.69	1.09	0.12	3.45	0.1	0.2449	0.0270	0.033	0.0074	0.0303
SA-1	14	-3.2	0.68	1.05	1.54	0.45	0.08	1.448	0.09	0.1838	0.0327	0.016	0.0065	0.0356
SA-2	18	-7.0	0.77	0.90	1.17	0.79	0.12	2.353	0.17	0.2356	0.0358	0.016	0.0048	0.0203
RW-1	14	-2.5	0.78	1.00	1.28	0.89	0.16	2.935	0.07	0.2262	0.0407	0.002	0.0005	0.0022
RW-1	23	-11.5	1.03	1.85	1.80	0.1	0.01	0.541	0.17	0.0649	0.0065	0.001	0.0006	0.0100
RW-2	18	-6.5	1.14	0.86	0.75	0.94	0.17	2.929	0.05	0.2392	0.0433	0.021	0.0053	0.0223
RW-3	12	-1.2	0.78	1.15	1.47	0.36	0.05	1.253	0.11	0.1598	0.0222	0.02	0.0089	0.0556
RW-3	20	-9.2	1.04	1.05	1.01	0.56	0.08	1.673	0.03	0.2095	0.0299	0.029	0.0108	0.0518
RW-4	14	-3.1	0.68	1.15	1.69	0.43	0.06	1.501	0.01	0.1719	0.0240	0.013	0.0052	0.0302
RW-4	20	-9.1	0.85	0.95	1.12	0.71	0.11	2.05	0.34	0.2328	0.0361	0.016	0.0052	0.0225
				Average	**1.3731**	**0.6200**	**0.0945**	**2.0034**	**0.1118**	**0.1946**	**0.0296**	**0.0167**	**0.0056**	**0.0286**
				Average (Abutment)	**1.4947**	**0.7075**	**0.1000**	**2.2888**	**0.1125**	**0.2091**	**0.0307**	**0.0205**	**0.0061**	**0.0300**
				Average (RW-1 to RW-4)	**1.3036**	**0.5700**	**0.0914**	**1.8403**	**0.1114**	**0.1863**	**0.0289**	**0.0146**	**0.0052**	**0.0278**

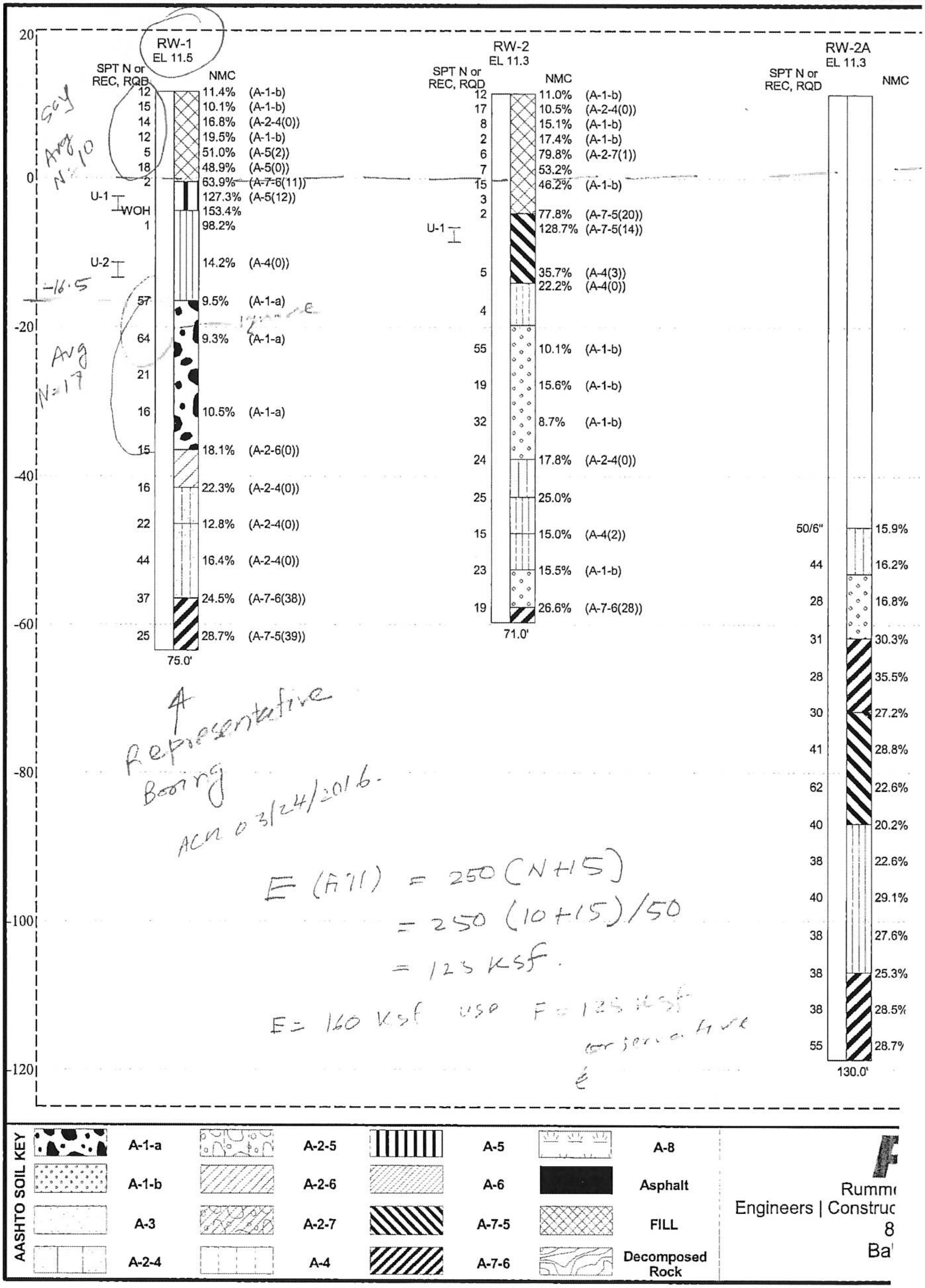
$CC = \frac{C_c}{1+e_o}$ $CR = \frac{C_r}{1+e_o}$

East Bank (Strata Ia)

Boring No.	Depth	Elevation	Overburden	P_c	OCR	C_c	C_r	e_o	Cv	CC	CR	C_α	C_α'	
AB-1	12	-6.15	0.72	0.60	0.8	0.38	0.05	1.631	0.06	0.1444	0.0190	0.006	0.0023	0.0158
AB-2	12	-6.89	0.55	0.85	1.5	0.33	0.03	1.265	0.44	0.1457	0.0132	0.004	0.0018	0.0121
AB-3	20	-13.75	1.26	1.12	0.9	0.3	0.02	1.199	0.35	0.1364	0.0091	0.004	0.0018	0.0133
AB-4	14	-5.77	0.65	1.15	1.8	0.37	0.03	1.445	0.89	0.1513	0.0123	0.006	0.0025	0.0162
RW-5	16	-9.8	0.71	1.20	1.7	0.39	0.04	1.503	0.04	0.1558	0.0160	0.023	0.0092	0.0590
RW-5	30	-23.8	1.11	0.82	0.7	0.4	0.06	1.758	0.01	0.1450	0.0218	0.011	0.0040	0.0275
RW-6	12	-7.4	0.44	0.78	1.8	0.42	0.03	1.627	0.16	0.1599	0.0114	0.022	0.0084	0.0524
RW-7	10	-4.68	0.50	0.65	1.3	0.61	0.08	2.062	0.1	0.1992	0.0261	0.04	0.0131	0.0656
RW-8	8	-3.13	0.54	0.80	1.5	0.52	0.05	1.791	0.2	0.1863	0.0179	0.006	0.0021	0.0115
RW-8	18	-13.13	0.67	0.82	1.2	0.67	0.09	2.243	0.18	0.2066	0.0278	0.008	0.0025	0.0119
				Average	**1.3244**	**0.4390**	**0.0480**	**1.6524**	**0.2430**	**0.1631**	**0.0175**	**0.0130**	**0.0048**	**0.0285**
				Average (Abutment)	**1.2592**	**0.3450**	**0.0325**	**1.3850**	**0.4350**	**0.1445**	**0.0134**	**0.0050**	**0.0021**	**0.0144**
				Average (RW-5 to RW-8)	**1.3678**	**0.5017**	**0.0583**	**1.8307**	**0.1150**	**0.1755**	**0.0202**	**0.0183**	**0.0065**	**0.0380**

Potomac Clay

Boring No.	Depth	Elevation	Overburden	P_c	OCR	C_c	C_r	e_o	Cv	CC	CR	C_α	C_α'	
W-4	51	-52.5	1.50	0.70	0.47	0.19	0.07	0.802	0.01	0.1054	0.0388	0.0110	0.0061	0.0579



Say
Avg N=10

-16.5

ignore

Avg N=17

Representative Boring

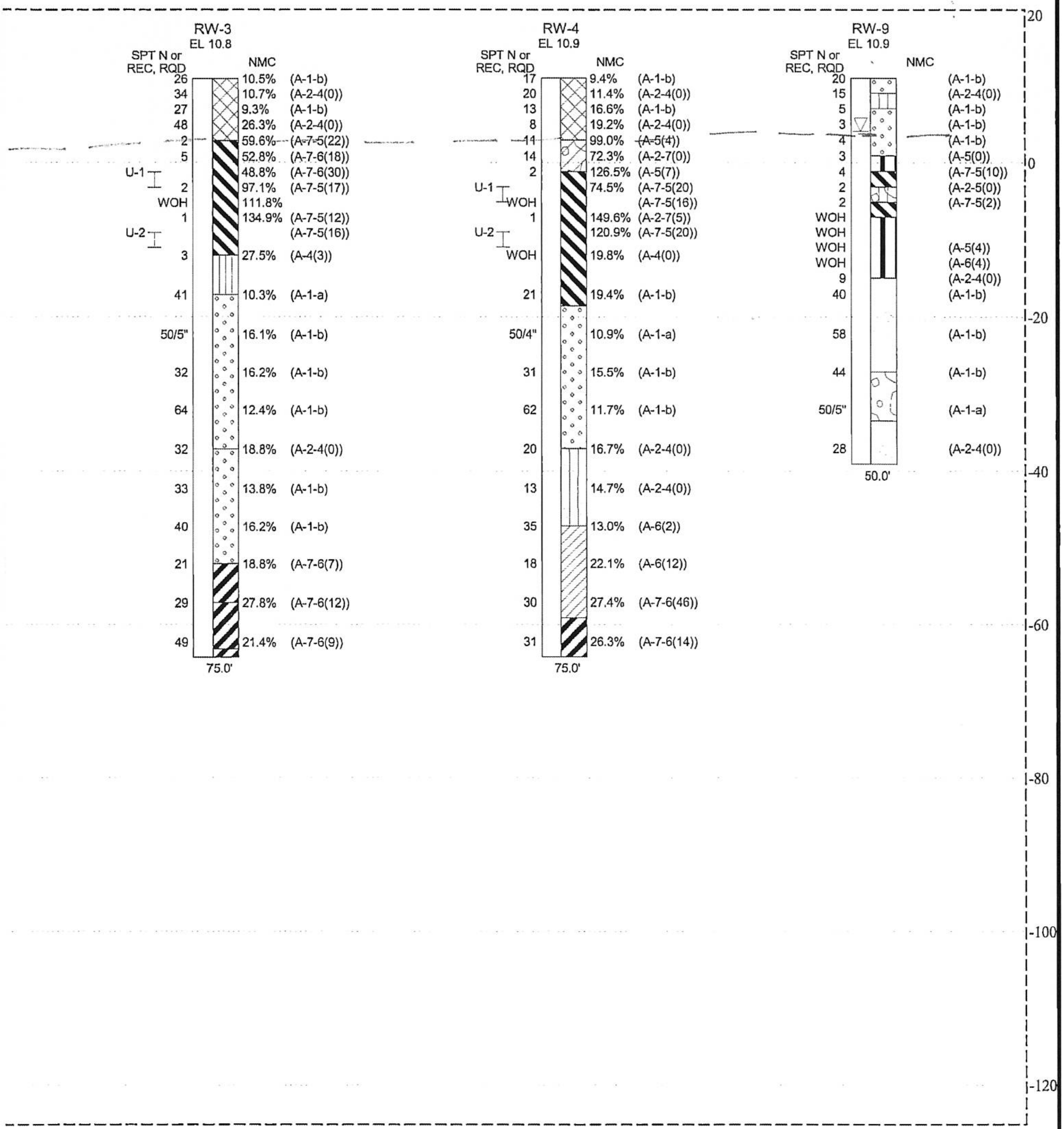
ACN 03/24/2016.

$E(A11) = 250(N+15)$
$= 250(10+15)/50$
$= 125$ ksf.

E = 160 ksf use E = 125 ksf
conservative
é

RKK FENCE - AASHTO CRISTINA RIVER BRIDGE.GPJ RKK_CURRENT.GDT 2/17/14

(4)



& Kahl, LLP ers | Planners | Scientists Street) 21217 2900	Title: **Summary of Boring Data** **West Approach Ramp**	Figure No. **A-3e**
	Drawn: ACR | Approved: EMK | Date: 2/17/14	Comm. No. 104-130

(5)

 BOWLES, 4th Ed

TABLE 5-5 Equations for stress-strain modulus E_s by several test methods

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf. The N values should be estimated as N_{55} and not N_{70}

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15\,000 \text{ to } 22\,000) \ln N$ $E_s§ = (35\,000 \text{ to } 50\,000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s† = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	
Sand (overconsolidated)	$E_s‡ = 18\,000 + 750N$ $E_{s(OCR)} = E_{s(nc)}\,(\text{OCR})^{1/2}$	$E_s = 6 \text{ to } 30q_c$
Gravelly sand and gravel	$E_s = 1200(N + 6)$ $E_s = 600(N + 6) \quad N \le 15$ $E_s = 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Soft clay	—	$E_s = 3 \text{ to } 8q_c$
	Using the undrained shear strength s_u in units of s_u	
Clay	$I_P > 30$ or organic	$E_s = 100 \text{ to } 500s_u$
	$I_P < 30$ or stiff	$E_s = 500 \text{ to } 1500s_u$
	$E_{s(OCR)} = E_{s(nc)}\,(\text{OCR})^{1/2}$	

† Vesić (1970).
‡ Author's equation from plot of D'Appolonia et al. (1970).
§ USSR (and may not be standard blow count N).
General sources: European Conference on Standard Penetration Testing (1974), vol. 2.1, pp. 150–151; CGJ, November 1983, pp. 726–737; Use of In Situ Tests in Geotechnical Engineering, ASCE (1986), p. 1173; Mitchel and Gardner (1975).

E (5'11) = SEE TABLE 2-7 P 99

SHALE OIL $E_{(psi)} = 1500$ (CBR)

CLAYS

	s_u (ksf)
HARD	>4
VERY STIFF	2–4
STIFF	1–2
MEDIUM	0.5–1
SOFT	0.25–0.5
VERY SOFT	0–0.25

TRIAXIAL $E = 2.5 q_c$

PLANE-STRAIN $E = 3.5 q_c$

pg 99 Bowles

TABLE 2-7 Typical range of values for the static stress-strain modulus E_s for selected soils

Field values depend on stress history, water content, density, etc.

Soil	E_s ksf	E_s Mpa
Clay		
Very soft	50–250	2–15
Soft	100–500	5–25
Medium	300–1000	15–50
Hard	1000–2000	50–100
Sandy	500–5000	25–250
Glacial till		
Loose	200–3200	10–150
Dense	3000–15 000	150–720
Very dense	10 000–30 000	500–1440
Loess	300–1200	15–60
Sand		
Silty	150–450	5–20
Loose	200–500	10–25
Dense	1000–1700	50–81
Sand and gravel		
Loose	1000–3000	50–150
Dense	2000–4000	100–200
Shale	3000–300 000	150–5000
Silt	40–400	2–20

TABLE 2-8 Values or value ranges for Poisson's ratio μ

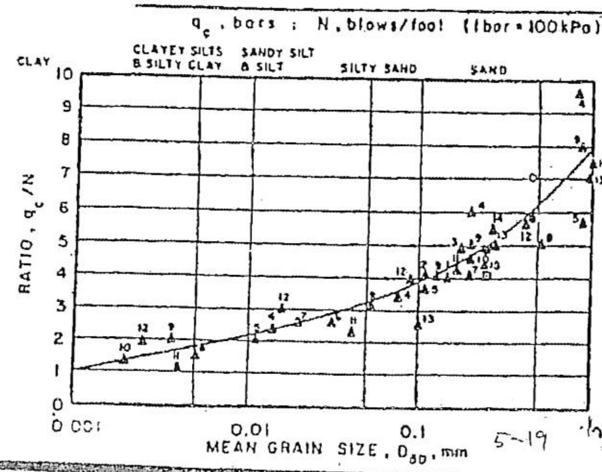
μ FOR SAND $= \dfrac{1 - \sin\phi}{2 - \sin\phi}$

$\mu = 0.1 + 0.3\,\phi_r$

$\phi_r = \dfrac{\phi - 25°}{45° - 25°}$

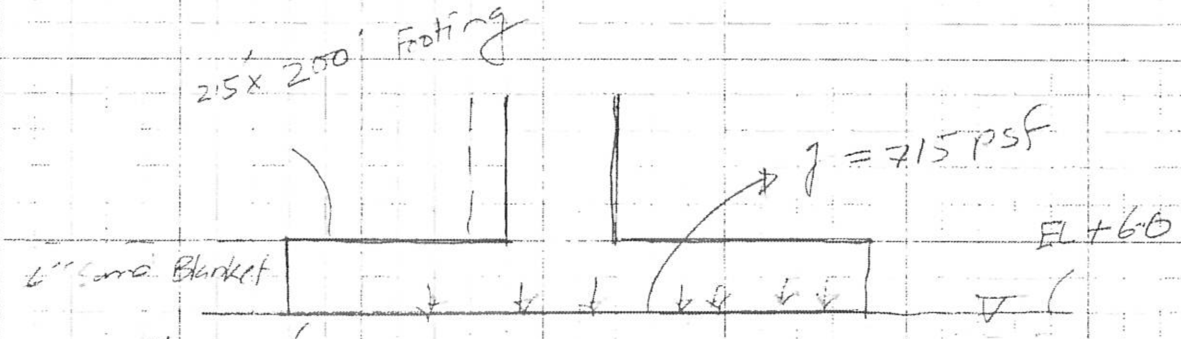
FANG p 590

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	−0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15



5-19

Soil Profile and Parameters:



(Fill) $\gamma = 120$ pcf (Granular Soil)
$\phi = 27°$
$E = 125$ ksf ($N = 15$ BPF)
$\mu = 0.30$

EL +0.0

(STR Ia) $\gamma = 115$ pcf, $S_u = 250$ psf, OCR = 1.30
$E = 150$ ksf, $\mu = 0.40$ (cohesive soil)

Consolidating Layer $\frac{C_c}{1+e_0} = 0.186$, $\frac{C_r}{1+e_0} = 0.029$, $C_v = 0.1114$ ft²/day
$\frac{C_\alpha}{C_c} = 0.0780$

EL -16.5

(STR. Ib) $\gamma = 125$ pcf, $\mu = 0.30$
$E = 125$ ksf. (Granular Soil)

Subsurface Profile.

Christina River Bridge

Report created by FoSSA(2.0): Copyright (c) 2003-2012, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title: Christina River Bridge
Project Number: 04130 - 03G
Client: Delaware DOT
Designer: ACR ACR 03/24/2016.
Station Number:
ENM 4/20/16

Description:
Settlement Analysis of Grade Beam Supporting EPS Facing Panel

Company's information:

Name: RK&K LLP
Street: 81 Mosher Street
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Original file path and name: M:\project ..... esign Calcs 2016\Grade Beam\Grade Beam support.2ST
Original date and time of creating this file: Wed Mar 23 10:55:33 2016

GEOMETRY: Analysis of a Multiple Footings

INPUT DATA – FOUNDATION LAYERS – 3 layers

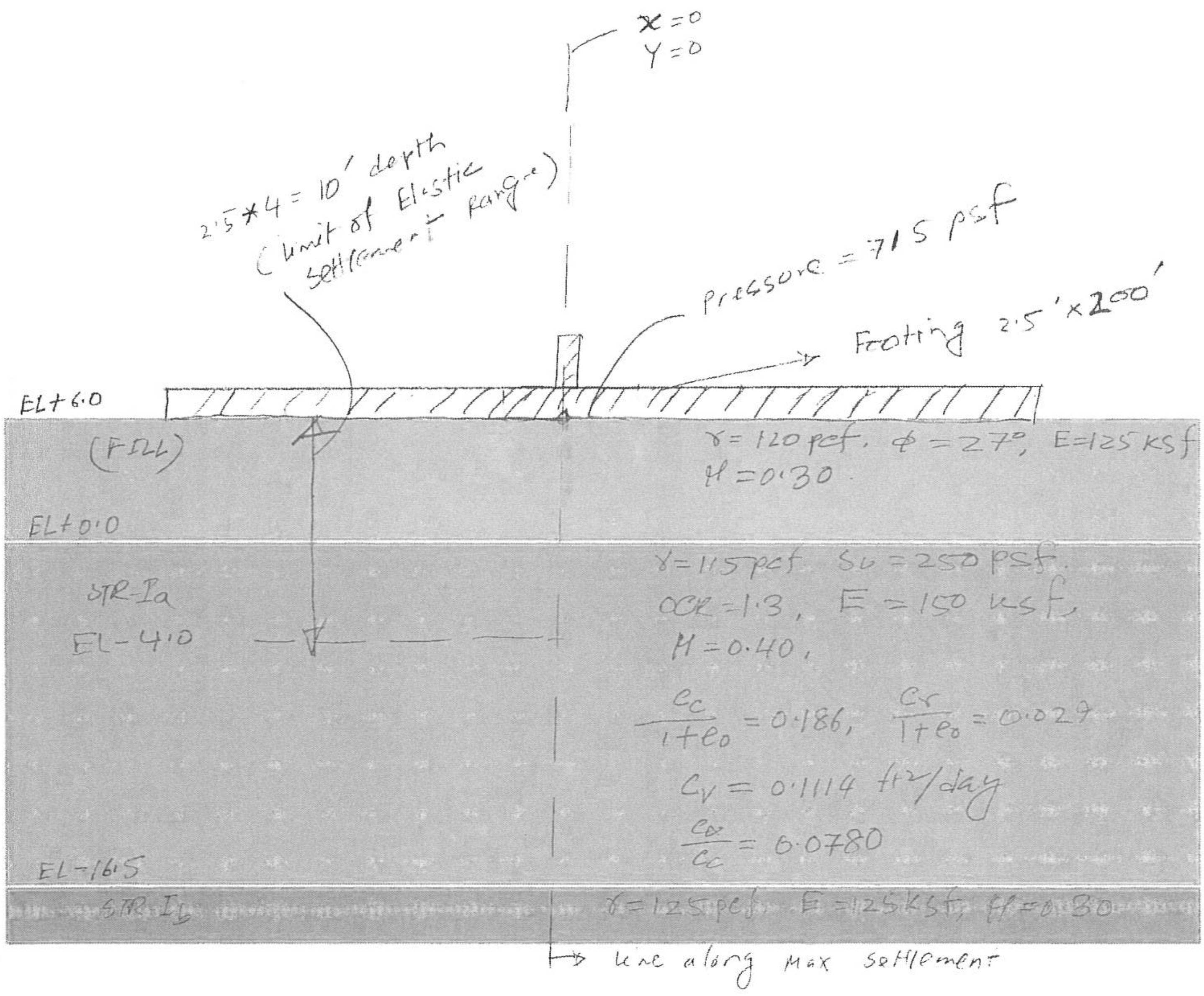
	Wet Unit Weight, γ [lb/ft^3]	Poisson's Ratio μ	Description of Soil
1	120.00	0.30	FILL
2	115.00	0.40	Cohesive (STR Ia)
3	125.00	0.30	Cohesionless (STR-Ib)

INPUT DATA – MULTIPLE FOOTINGS – 1 footings

INPUT DATA OF WATER

Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]
1	-100.00	6.00
2	100.00	6.00

DRAWING OF SPECIFIED GEOMETRY



 10

INPUT DATA FOR CONSOLIDATION — $\alpha = 1/6$

Layer #	Underging Consolidation [Yes/No]	OCR = Pc / Po	Cc / (1+e0)	Cr / (1+e0)	e0	Cv [ft ^{2}/day]	Drains at :
1	No	N/A	N/A	N/A	N/A	N/A	N/A
2	Yes	1.30	0.186	0.029	N/A	0.1114	Top & Bot.
3	No	N/A	N/A	N/A	N/A	N/A	N/A

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IMMEDIATE SETTLEMENT, Si

Node #	Settlement along section: X [ft.]	Y [ft.]	Layer (k)	Young's Modulus, E [lb/ft ²]	Poisson's Ratio, μ	Settlement of each layer, Si(k) [ft.]	Initial Z [ft.]	Final Z * [ft.]	Total Settlement Sum of Si(k), [ft.]
1	0.00	0.00	1	125000	0.3000	0.0161	6.00	5.98	0.02
			2	150000	0.4000	0.0034			
			3	125000	0.3000	0.0000			

*Note: Final Z is calculated assuming only 'Immediate Settlement' exists.

total Elastic S = 0.02 ft
= 0.24 in

ULTIMATE SETTLEMENT, Sc

Node #	X [ft.]	Y [ft.]	Original Z [ft.]	Settlement Sc [ft.]	Final Z * [ft.]
1	0.00	0.00	6.00	0.03	5.97

*Note: Final Z is calculated assuming only 'Ultimate Settlement' exists.

Consolidation. $S_c = 0.03 \times 12$
$= 0.36$ in

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TABULATED GEOMETRY: INPUT OF FOUNDATION SOILS

Found. Soil #	Point #	Coordinates (X, Z) : (X) [ft.]	(Z) [ft.]	DESCRIPTION
1	1	-100.00	6.00	FILL
	2	100.00	6.00	
2	1	-100.00	0.00	Cohesive (STR Ia)
	2	100.00	0.00	
3	1	-100.00	-16.50	Cohesionless (STR-Ib)
	2	100.00	-16.50	

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TABULATED GEOMETRY: INPUT OF MULTIPLE FOOTINGS – 1 footings

Footing #	L (in X direction) [ft]	B (in Y direction) [ft]	X of center [ft]	Y of center [ft]	q [ft]l [lb/ft²]
1	200.00	2.50	0.00	0.00	715.00

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BBS 4/21/16

EMK 4/26/16

RK&K

by: BBS Date: 4/21/2016

chk: Date:

SUBJECT: Christina River Bridge, Leveling Pad for EPS wall

RK&K Commision. No.: 105-053-02.GEO

REFERENCE:
EM 1110-1-1904
Schmertmann
Procedure

ELASTIC SETTLEMENT ANALYSIS

Settlement: **0.17 in**

Groundwater Depth = 6.0 ft

Creep Time = 75.00 yrs

L/B= 80.00

FOOTING WIDTH = 2.5 ft

FOOTING LENGTH = 200.0 ft

GROSS BEARING PRESSURE = 0.36 tsf

Effective Overburden 0.0 ft @ $\gamma_{overburden}$ = 120.0 pcf q = 0.00 tsf

NET BEARING PRESSURE Existing Foundation Surcharge = 0.00 tsf Δq = 0.36 tsf

INFLUENCE COEFFICIENTS

LIMITING VALUE OF I =0.6

L/B=1 Effective Stress @ B/2

γ_{v1} = 120 pcf 1.3 ft dv= 0.08 tsf

γ_{v2} = 57.6 pcf -4.8 ft dv= 0.00 tsf

σ'_{Izp} @ depth = B/2 0.08 TSF

I_{zp}= 0.7183

L/B=10 Effective Stress @ B

γ_{v1} = 120 pcf 2.5 ft dv= 0.15 tsf

γ_{v2} = 57.6 pcf -3.5 ft dv= 0.00 tsf

σ'_{Izp} @ depth = B 0.15 TSF

I_{zp}= 0.6544

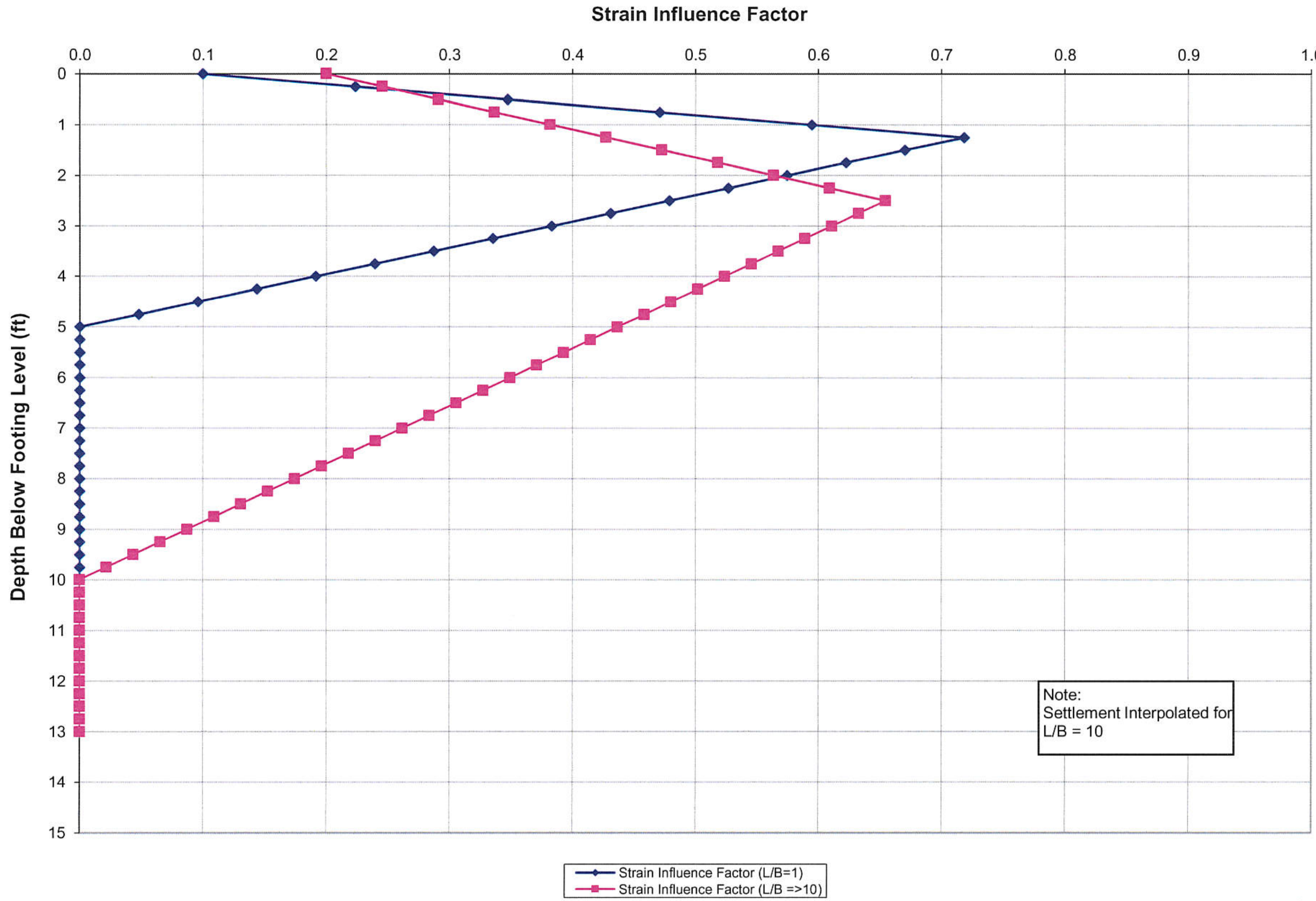
FOUNDING ELEVATION 6

Depth step value = 0.3 ft

ELEV	DEPTH FT	Z/B	I_z	I_z	0.7183 I_z (L/B)=1	I_z	I_z	0.6544 I_z (L/B)=10	E_s (TSF)	L/B=1 (I*D)/E	L/B=10 (I*D)/E
6.0	0.00	0.000	0.100	0.000	0.1000	0.200	0.000	0.2000	125.00	0.0002	0.0004
5.8	0.25	0.100	0.224	0.000	0.2237	0.245	0.000	0.2454	125.00	0.0004	0.0005
5.5	0.50	0.200	0.347	0.000	0.3473	0.291	0.000	0.2909	125.00	0.0007	0.0006
5.3	0.75	0.300	0.471	0.000	0.4710	0.336	0.000	0.3363	125.00	0.0009	0.0007
5.0	1.00	0.400	0.595	0.000	0.5947	0.382	0.000	0.3818	125.00	0.0012	0.0008
4.8	1.25	0.500	0.718	0.000	0.7183	0.427	0.000	0.4272	125.00	0.0014	0.0009
4.5	1.50	0.600	0.000	0.670	0.6704	0.473	0.000	0.4726	150.00	0.0011	0.0008
4.3	1.75	0.700	0.000	0.623	0.6226	0.518	0.000	0.5181	150.00	0.0010	0.0009
4.0	2.00	0.800	0.000	0.575	0.5747	0.564	0.000	0.5635	150.00	0.0010	0.0009
3.8	2.25	0.900	0.000	0.527	0.5268	0.609	0.000	0.6089	150.00	0.0009	0.0010
3.5	2.50	1.000	0.000	0.479	0.4789	0.654	0.000	0.6544	150.00	0.0008	0.0011
3.3	2.75	1.100	0.000	0.431	0.4310	0.000	0.633	0.6326	150.00	0.0007	0.0011
3.0	3.00	1.200	0.000	0.383	0.3831	0.000	0.611	0.6108	150.00	0.0006	0.0010
2.8	3.25	1.300	0.000	0.335	0.3352	0.000	0.589	0.5889	150.00	0.0006	0.0010
2.5	3.50	1.400	0.000	0.287	0.2873	0.000	0.567	0.5671	150.00	0.0005	0.0009
2.3	3.75	1.500	0.000	0.239	0.2394	0.000	0.545	0.5453	150.00	0.0004	0.0009
2.0	4.00	1.600	0.000	0.192	0.1916	0.000	0.524	0.5235	150.00	0.0003	0.0009
1.8	4.25	1.700	0.000	0.144	0.1437	0.000	0.502	0.5017	150.00	0.0002	0.0008
1.5	4.50	1.800	0.000	0.096	0.0958	0.000	0.480	0.4799	150.00	0.0002	0.0008
1.3	4.75	1.900	0.000	0.048	0.0479	0.000	0.458	0.4581	150.00	0.0001	0.0008
1.0	5.00	2.000	0.000	0.000	0.0000	0.000	0.436	0.4363	150.00	0.0000	0.0007
0.8	5.25	2.100	0.000	0.000	0.0000	0.000	0.414	0.4144	150.00	0.0000	0.0007
0.5	5.50	2.200	0.000	0.000	0.0000	0.000	0.393	0.3926	150.00	0.0000	0.0007
0.3	5.75	2.300	0.000	0.000	0.0000	0.000	0.371	0.3708	150.00	0.0000	0.0006
0.0	6.00	2.400	0.000	0.000	0.0000	0.000	0.349	0.3490	150.00	0.0000	0.0006
-0.3	6.25	2.500	0.000	0.000	0.0000	0.000	0.327	0.3272	150.00	0.0000	0.0005
-0.5	6.50	2.600	0.000	0.000	0.0000	0.000	0.305	0.3054	150.00	0.0000	0.0005
-0.8	6.75	2.700	0.000	0.000	0.0000	0.000	0.284	0.2836	150.00	0.0000	0.0005
-1.0	7.00	2.800	0.000	0.000	0.0000	0.000	0.262	0.2618	150.00	0.0000	0.0004
-1.3	7.25	2.900	0.000	0.000	0.0000	0.000	0.240	0.2399	150.00	0.0000	0.0004
-1.5	7.50	3.000	0.000	0.000	0.0000	0.000	0.218	0.2181	150.00	0.0000	0.0004
-1.8	7.75	3.100	0.000	0.000	0.0000	0.000	0.196	0.1963	150.00	0.0000	0.0003
-2.0	8.00	3.200	0.000	0.000	0.0000	0.000	0.175	0.1745	150.00	0.0000	0.0003
-2.3	8.25	3.300	0.000	0.000	0.0000	0.000	0.153	0.1527	150.00	0.0000	0.0003
-2.5	8.50	3.400	0.000	0.000	0.0000	0.000	0.131	0.1309	150.00	0.0000	0.0002
-2.8	8.75	3.500	0.000	0.000	0.0000	0.000	0.109	0.1091	150.00	0.0000	0.0002
-3.0	9.00	3.600	0.000	0.000	0.0000	0.000	0.087	0.0873	150.00	0.0000	0.0001
-3.3	9.25	3.700	0.000	0.000	0.0000	0.000	0.065	0.0654	150.00	0.0000	0.0001
-3.5	9.50	3.800	0.000	0.000	0.0000	0.000	0.044	0.0436	150.00	0.0000	0.0001
-3.8	9.75	3.900	0.000	0.000	0.0000	0.000	0.022	0.0218	150.00	0.0000	0.0000
-4.0	10.00	4.000	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-4.3	10.25	4.100	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-4.5	10.50	4.200	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-4.8	10.75	4.300	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-5.0	11.00	4.400	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-5.3	11.25	4.500	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-5.5	11.50	4.600	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-5.8	11.75	4.700	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-6.0	12.00	4.800	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-6.3	12.25	4.900	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-6.5	12.50	5.000	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-6.8	12.75	5.100	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
-7.0	13.00	5.200	0.000	0.000	0.0000	0.000	0.000	0.0000	150.00	0.0000	0.0000
									SUM-ft/tsf	0.0133	0.0243
									C1=	1.0000	1.0000
									C2=	1.6000	1.6000
									Δq =	0.3575	0.3575
										0.008	0.014
									ë=	0.09	0.17

FOR L/B= 1 0.09 INCHES

I-95 MD43 Interchange
King Avenue Bridge Replacement



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PURPOSE - BEARING RESISTANCE FOR GRADE BEAM SUPPORTING EPS FACING PANEL

ASSUMPTIONS.

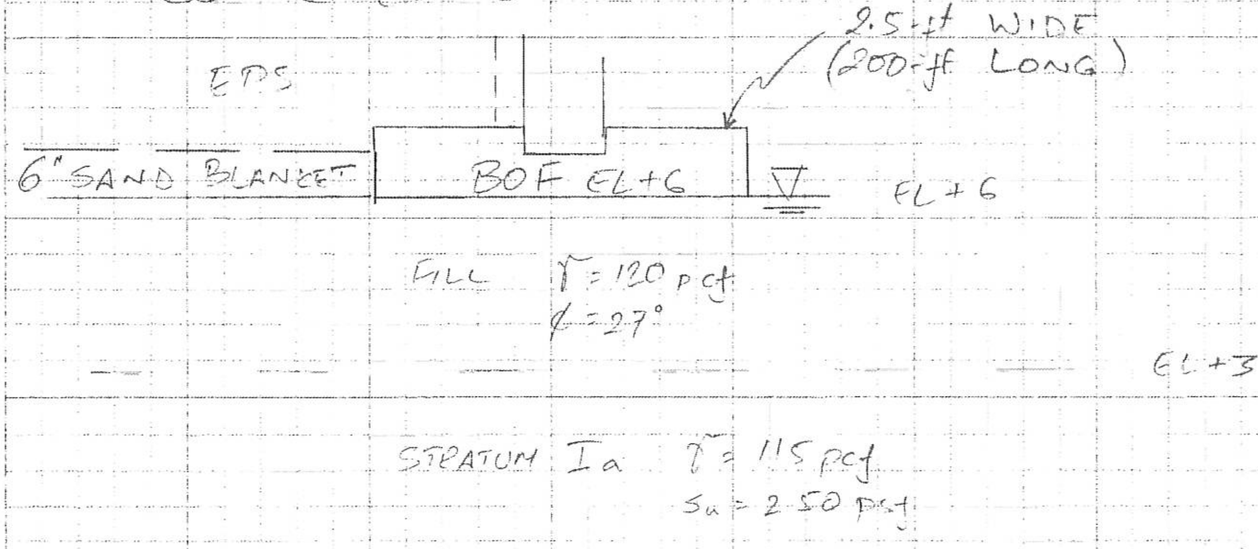
1) THE GRADE BEAM WILL BE FOUNDED ON EXISTING FILL OVER SOFT STRATUM Ia

2) CHECK BEARING RESISTANCE FOR 2-ft WIDE, 2.5-ft & 3-ft WIDE FOOTINGS.

3) ASSUME NO DEPTH OF EMBEDMENT = 0.5 ft ONLY.

SOIL PROFILE

BOF @ EL+6.0



NOMINAL BEARING RESISTANCE FOR LAYERED SYSTEM WITH THE UPPER LAYER IS COHESIONLESS

$$q_n = q_2 e^{0.67[1+B/L]H/B}$$

$B = 2, 2.5, 3.0$ $H = 3.0$ $L = \cancel{150} \; 200$ ft

q_2 = BEARING RESISTANCE IN STRATUM Ia

$S_u = 250$ psf

$$q_2 = c N_c S_c + \gamma D_f N_q S_q d_q C_q + 0.5 \gamma B N_\gamma S_\gamma C_\gamma$$

$c = 250$ psf $D_f = 0.5$ $N_\gamma = 0$

$N_c = 5.14$ $N_q = 1$

$S_c = 1 + \frac{B}{5L} = 1$ $S_q = 1$

$d_q = 1.0$

$C_q = 0.5$

B	BEARING RESISTA q_2	q_n
2.0	1315 psf	3665
2.5	1315 psf	~~2977~~ 2970 B = 200'
3.0	1315 psf	2604

USE $q_n = 2500$ psf

FOR ASD $q_{ULT} = 2500$ psf

RK&K
81 Mosher Street
Baltimore, Maryland 21217

GEOTECHNICAL QUALITY CONTROL SIGN OFF SHEET

Project Name: Christina River Bridge

Commission Number: 04-130-03G

Client Name: DelDOT

Document Title: Design of Load Transfer Platform for Deep Mixing Method

Document Type: (Check One)
- ☐ Proposal
- ☐ Report/Memo
- ☒ Calculations
- ☐ Boring Logs
- ☐ Specification
- ☐ Other: ______

Project Milestone: ______ **Due Date:** ______

This is to certify that I was the originator of this work product, and that I have carefully checked my work prior to passing it on for Quality Control checking.

Printed Name: Bibek Shrestha

Signature: Bibek Shrestha **Date:** 4/14/2016

	Print Name	Signature	Date
☑ Checked:	ERIC KLEIN	Eh	4/20/16
☑ Corrected:	BIBEK SHRESTHA	Bibek	4/21/16
☐ Checked:			
☐ Corrected:			
☐ Checked:			
☐ Project Engr:			
☑ PM/Manager:	ERIC M KLEIN	EMKlein	4/26/16

RK&K
81 Mosher Street
Baltimore, Maryland 21217

GEOTECHNICAL CALCULATION QUALITY CONTROL CHECKLIST

Project: Christina River Bridge **Comm. No:** 04-130-03G

Title of Calculation Set: Design of Load Transfer Platform for Deep Mixing Method

RK&K Project Manager: EMK **Coordinating Discipline POC:** ____________

Calculations Prepared By: BBS **Design Method/Code:** ____________

File Path on Network: ____________

Subsurface Characterization Approved By: ____________

Initial Calculations: Signature of Originator: Bibek Shrestha Date: 4/14/2016

- ☒ Geotech Quality Control Sign Off Sheet is provided.
- ☒ Calculations are neat, legible and understandable.
- ☒ Variable inputs are highlighted and check confirmations are clearly marked in calculation spreadsheets.
- ☒ Table of Contents is provided.
- ☒ Heading on each page filled out, including computer input and output.
- ☒ The first page contains purpose of the calculation.
- ☒ References for design criteria, data, methods, formulas and computer program names and version numbers are given, and included in appendices as necessary.
- ☒ All assumptions are listed, explained, and those needed to be verified later are flagged.
- ☐ All input by others is attached and includes originator, date provided. (i.e. Structures loads, concrete/steel properties, etc.)
- ☒ Can the analytical steps involved be followed independent of the originator?
- ☒ Do the calculations agree with the other inter-department / project documents?
- ☐ Footer with file path/tab name and print date is provided on calculation spreadsheets.
- ☐ Detailed CAD ready sketches are provided that clearly depict the design requirements.
- ☒ Calculation Summary with recommendations included in front of calculation set.
- ☒ All calculations are checked and each calculation sheet is initialed by the originator before giving to the checker.

Finalized Calculations: Signature of Originator: ____________ Date: ____________

- ☐ All pages are numbered sequentially and cross-references are complete and accurate.
- ☐ All calculations are checked and each calculation sheet initialed by the checker.
- ☐ Calculation set reviewed by Project Manager.
- ☐ Has this calculation been superseded? Reference new calculation, if applicable.

This sheet is to be prepared by the originator of the calculation and attached to the front of the calculation set

RK&K	Subject CHRISTINA RIVER BRIDGE	Page i of
	LTP REINFORCEMENT DESIGN	Cm. No. 04-130-036
	Prepared By BBS Date 4/14/16 Checked By GM	Date 4/20/16

TABLE OF CONTENT

SUMMARY.

LTP THICKNESS MINIMUM $\left(\frac{s-d}{2}\right)$

LTP CONSTRUCTED USING GRADED AGGREGATE TYPE B CRUSHER RUN

USE A MINIMUM OF 3 LAYERS OF GEOGRID ~~SUMMARY~~.

@ 5% STRAIN FOR BOTH (s-d) = 5 ft & 8-ft
CLEAR SPACING BETWEEN COLUMN

USE BIAXIAL GEOGRID WITH TENSILE STRENGTH > 386 lb/ft

TENSAR BIAXIAL ~~1100~~ BX1500 WORKS.

TENSAX TRIAXIAL TX GEOGRID ALSO WORKS.

PURPOSE: DESIGN OF LOAD TRANSFER PLATFORM FOR DEEP MIXING METHOD.

DESIGN REQUIREMENT

1> THICKNESS OF LTP ≥ ONE HALF THE CLEAR SPAN BETWEEN COLUMNS.
2> MINIMUM THREE LAYERS OF REINFORCEMENT
3> MINIMUM 6-INCHES BETWEEN REINFORCEMENT
4> USE GRADED AGGREGATE BASE FOR LTP.

D
DE
F

REFERENCE:

GEOSYNTHETIC REINFORCED COLUMN SUPPORT EMBANKMENT DESIGN GUIDELINES BY COLLIN, HAN, HUANG.

USE BEAM (COLLIN) METHOD FOR DESIGN.

ASSUMPTION:

① ASSUME SQUARE PATTERN COLUMN SPACING

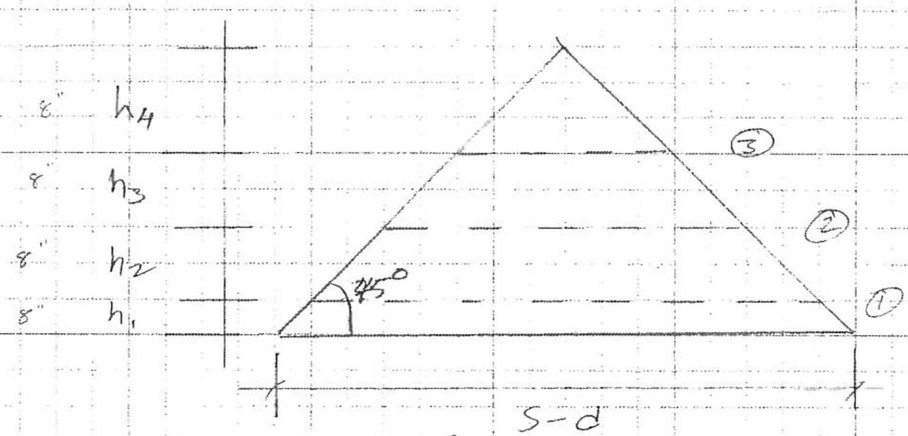
CLEAR SPACING BETWEEN COLUMNS (S-d) = 5 ft & 8 ft

② ANGLE OF ARCHING 45°

③ TENSILE LOAD IN REINFORCEMENT BASED ON TENSION MEMBRANE THEORY

④ INITIAL STRAIN IN REINFORCEMENT IS LIMITED TO 5%

h_n = 8-INCHES



DESIGN FOR LAYER ①

FOR S-d = 5 ft h_n = 8 in (EQUAL DIVISION)

D1

UNIFORM VERTICAL LOAD ON LAYER N REINFORCEMENT

$$W_{TN} = [A_N + A_{n+1}] h_n \gamma / 2A_n$$

A = AREA OF REINFORCEMENT LAYER n OR n+1

$$= [(S-d) - 2(\Sigma \text{ REINFORCEMENT VERTICAL SPACING}/\tan 45)]^2$$

$$A_1 = \left[5 - 2\left(\frac{8}{12} \times \frac{1}{\tan 45}\right)\right]^2$$

$$A_1 = \underline{\underline{13.44 \text{ ft}^2}}$$

$$A_2 = \left[5 - 2\left(\frac{16}{12} \times \frac{1}{\tan 45}\right)\right]^2$$

$$A_2 = \underline{\underline{5.44 \text{ ft}^2}}$$

h_n = 8 in = 0.67 ft

γ = 130 pcf GRADED AGGREGATE

$$W_{TN_{①}} = (13.44 + 5.44) \times \frac{0.67 \times 130}{2 \times 13.44} = \underline{\underline{61.18 \text{ lb/ft}^2}}$$

TENSILE LOAD IN REINFORCEMENT

$T_{rpt①} = W_{TN}\ \Omega\ D/2$

Ω = DIMENSIONLESS FACTOR (SEE PAGE 7 of 8)

Ω	REINFORCEMENT STRAIN (ε)%
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5

D = DESIGN SPAN FOR TENSIONED MEMBRANE

$= 1.41 \times [(S-d) - 2(\Sigma \text{VERTICAL SPACING}/\tan 45)]$

$D_1 = 1.41 \times [5 - 2 \times (8/12 \times \frac{1}{\tan 45})]$

$= \underline{5.17\ ft}$

$T_{rpt①} = W_{TN①}\ \Omega\ D/2$

@ $\varepsilon = 1\%$ $= 61.18 \times 2.07 \times \frac{5.17}{2} = 327.4$ lb/ft

@ $\varepsilon = 2\%$ $= 61.18 \times 1.47 \times 5.17/2 = 232.48$ lb/ft

@ $\varepsilon = 5\%$ $= 61.18 \times 0.97 \times \frac{5.17}{2} = 153.4$ lb/ft

TENSAR BX 1100 BIAXIAL GEOGRID

CHECK @ 5% STRAIN.

5% STRAIN TEN STR = 580 lb/ft OK

DESIGN FOR LAYER 2

$A_2 = 5.44\ ft^2$

$A_3 = \left[5 - 2 \times \left(\frac{24}{12} \times \frac{1}{\tan 45}\right)\right]^2$

$A_3 = \underline{\underline{1\ ft^2}}$ ✓

$W_{TN\,2} = (5.44 + 1) \times \frac{0.67 \times 130}{2 \times 5.44}$

$= \underline{\underline{51.55\ lb/ft^2}}$

$T_{rpn\,2} = W_{TN}\ \Omega\ D/2$

$D_2 = 1.41 \times \left[5 - 2 \times \left(\frac{16}{12} \times \frac{1}{\tan 45}\right)\right]$

$D_2 = \underline{3.29}$

$\varepsilon = 1\%$ $T_{rpn\,2} = 51.56 \times 2.07 \times 3.29/2 = 175.53\ lb/ft$

$\varepsilon = 2\%$ $= 51.55 \times 1.47 \times \frac{3.27}{2} = 124.65\ lb/ft$

$\varepsilon = 5\%$ $= 51.55 \times 0.97 \times \frac{3.29}{2} = 82.25\ lb/ft$

TENSAR BX 1100 BIAXIAL GEOGRID

FOR $S-d = 8\text{ ft}$ $h = 12\text{ in}$ equal DIVISION.

DESIGN FOR LAYER 1

$$W_{TN} = \left[A_N + A_{N+1}\right] \frac{h_n \gamma}{2A_N}$$

$$A_1 = \left[8 - 2\left(\frac{12}{12} \times \frac{1}{\tan 45}\right)\right]^2 = \underline{\underline{36\ ft^2}}$$

$$A_2 = \left[8 - 2\left[\frac{24}{12} \times \frac{1}{\tan 45}\right]\right]^2 = \underline{\underline{16\ ft^2}}$$

$h_n = 12\text{ in} = 1\text{ ft}.$

$\gamma = 130\text{ pcf}$

$$W_{TN\,1} = (36 + 16) \times \frac{1 \times 130}{2 \times 36} = 93.9\ lb/ft^2$$

$$D_1 = 1.41 \times \left[(S-d) - 2\left(\Sigma\,\text{VERTICAL SPACING}/\tan 45\right)\right]$$

$$= 1.41 \times \left[8 - 2 \times \left(\frac{12}{12} \times \frac{1}{\tan 45}\right)\right]$$

$$= \underline{\underline{8.46\ ft}}$$

$$T_{rpn\,1} = W_{TN\,1}\ \Omega\ \frac{D}{2}$$

$\varepsilon = 1\%$ $= 93.9 \times 2.07 \times 8.46/2 = 822.2\ lb/ft$

$\varepsilon = 2\%$ $= 93.9 \times 1.47 \times 8.46/2 = 583.9\ lb/ft$

$\varepsilon = 5\%$ $= 93.9 \times 0.97 \times 8.46/2 = 385.3\ lb/ft.$

TENSAR BX 1100 BIAXIAL GEOGRID

@ 5% STRAIN TEN STR = 580 lb/ft. OK

* GEOSYNTHETIC REINFORCED COLUMN SUPPORT EMBANKMENT DESIGN GUIDELINES COLLINS, HAN HUANG Pg 5

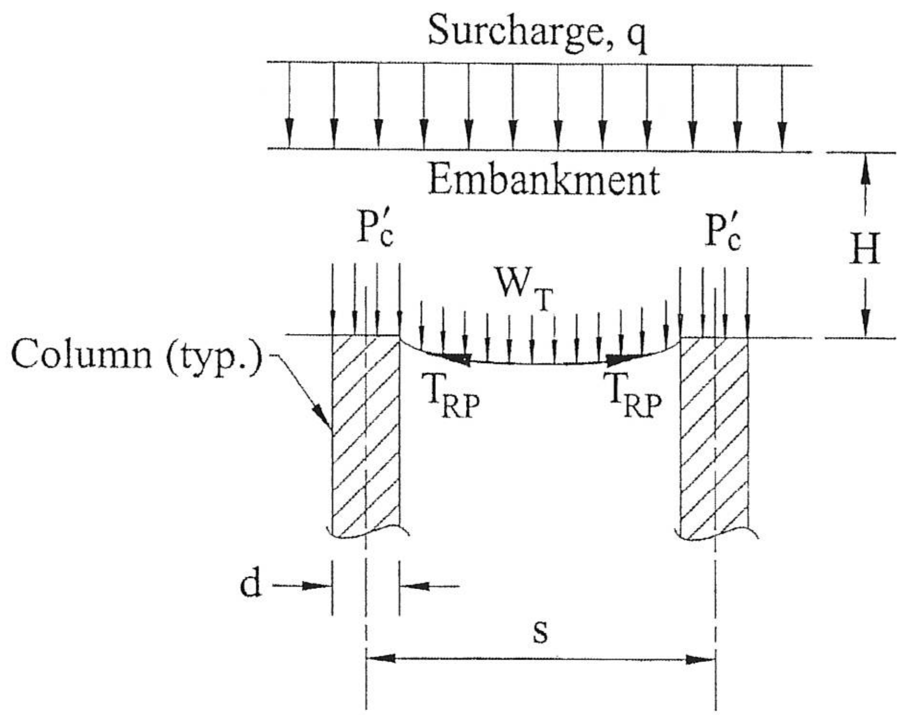


Figure 9. Definition of terms

6.2 Beam Method

The beam (Collin) method is based on the following assumptions:

- The thickness (h) of the load transfer platform is equal to or greater than one half the clear span between columns (s-d).
- A minimum of three layers of extensible (geosynthetic) reinforcement is used to create the load transfer platform.
- Minimum distance between layers of reinforcement is 150 mm (6 in.).
- Select fill is used in the load transfer platform.
- The primary function of the reinforcement is to provide lateral confinement of the select fill to facilitate soil arching within the height (thickness) of the load transfer platform.
- The secondary function of the reinforcement is to support the wedge of soil below the arch.
- The vertical load from the embankment above the load transfer platform is transferred to the columns below the platform.
- The initial strain in the reinforcement is limited to 5%.

The vertical load carried by each layer of reinforcement is a function of the column spacing pattern (*i.e.*, square or triangular) and the vertical spacing of the reinforcement. If the subgrade soil is strong enough to support the first lift of fill, the first layer of reinforcement is located 0.15-

Pg 7

0.25 m (6-10 in.) above subgrade. Each layer of reinforcement is designed to carry the load from the platform fill that is within the soil wedge below the arch. The fill load attributed to each layer of reinforcement is the material located between that layer of reinforcement and the next layer above (Figure 10).

The uniform vertical load on any layer (n) of reinforcement (W_{Tn}) may be determined from the equation below for an angle of arching of 45 degrees:

W_{Tn} = (area at reinforcement layer n + area at reinforcement layer (n+1))/2)
(layer thickness) (load transfer platform fill density)/(area at reinforcement layer n)

$$W_{Tn} = [A_n + A_{n+1}]\, h_n\, \gamma / 2\, A_n \qquad (5)$$

where: A = Area at reinforcement layer n or n+1
= $[(s\text{-}d) - 2(\Sigma \text{ReinforcementVertical Spacing}/\tan 45)]^2$ for square column spacing
= $[(s\text{-}d) - 2(\Sigma \text{ReinforcementVertical Spacing}/\tan 45)]^2 \sin 60/2$ for triangular column spacing

The tensile load in the reinforcement is determined based on tension membrane theory and is a function of the amount of strain in the reinforcement. The tension in the reinforcement is determined from the following equation:

$$T_{rpn} = W_{Tn}\, \Omega\, D/2 \qquad (6)$$

where: D = design span for tensioned membrane
= $1.41^*[(s\text{-}d) - 2(\Sigma \text{Vertical Spacing}/\tan 45)]$ for square column spacing
= $0.867^*[(s\text{-}d) - 2(\Sigma \text{Vertical Spacing}/\tan 45)]$ for triangular column spacing
Ω = dimensionless factor from tensioned membrane theory

Table 1. Values of Ω.

Ω	Reinforcement Strain (ε)%
2.07	1
1.47	2
1.23	3
1.08	4
0.97	5

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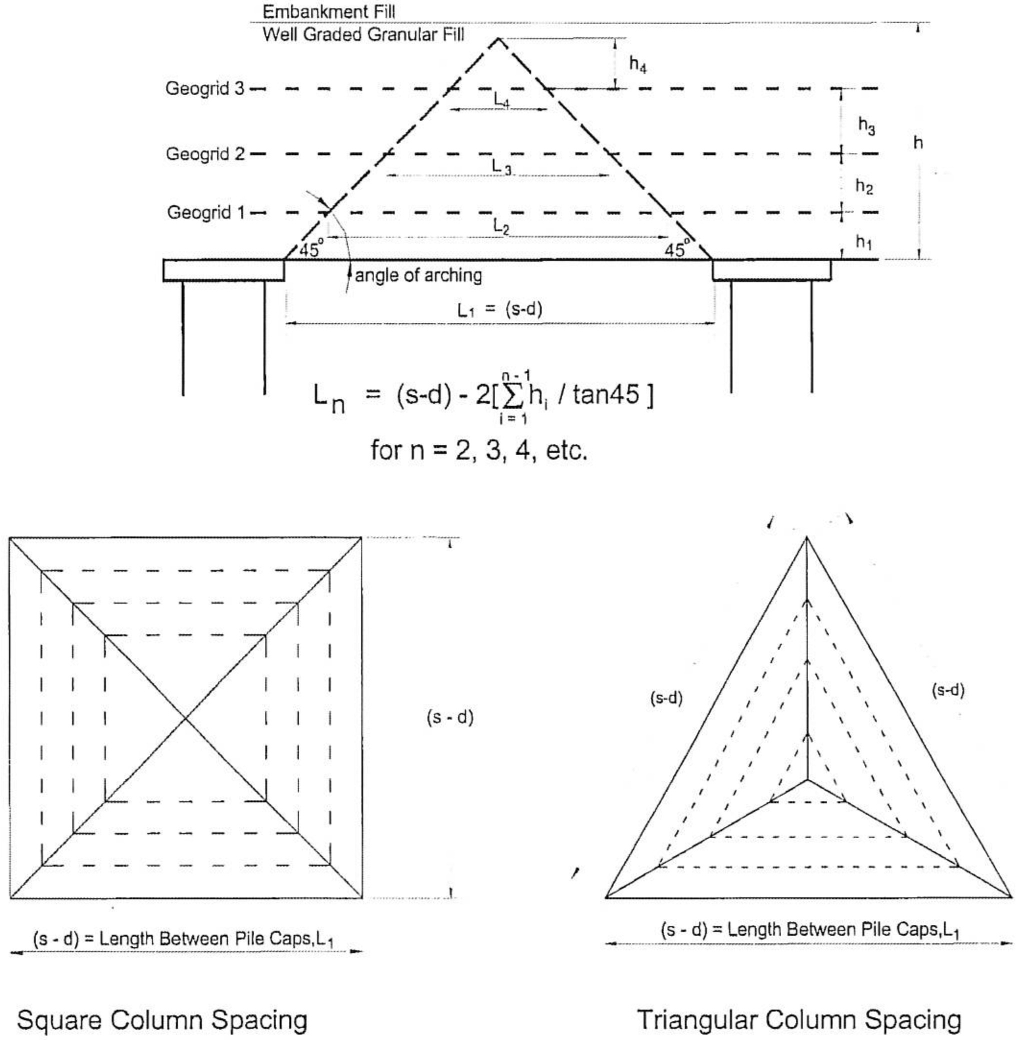

Figure 10. Load transfer platform design Collin method

CHECK GEOGRIDS FOR LONGTERM DESIGN STRENGTH

BX 1100 T_{ULT} = 850 lb/ft.

C_J JUNCTION EFF = 93%

C_I RESISTANCE TO INSTALLATION DAMAGE = 90% FOR GRADED AGGREGATE

C_D RESISTANCE TO LONG TERM DEGRADATION = 100%

FS_{CR} CREEP FACTOR OF SAFETY = 3.0

LONG TERM DESIGN STRENGTH (BX 1100) = $\dfrac{T_{ULT} \times C_J \times C_I \times C_D}{FS_{CR}}$

$= \dfrac{850 \times 0.93 \times 0.9 \times 1}{3}$

= 237.15 lb/ft < 386 lb/ft REQ 5% STRAIN FOR 8-ft SPACING

BX 1100 OK FOR 5-ft SPACING

BX 1500 T_{ULT} = 1850 lb/ft.

C_J = 93% C_I = 90% C_D = 100%

FS_{CR} = 3.0

LONG TERM DESIGN STRENGTH (BX1500) = $\dfrac{1850 \times 0.93 \times 0.9 \times 1}{3}$

= 516 lb/ft. > 386 lb/ft REQ @ 5% STRAIN

USE BX 1500.

ANY TRIAXIAL GRID

OTHER GRIDS W/ LTADS ≥ 386 CAN WORK, TOO.

Tx 130, 140, 160 WILL WORK

~~VERY HIGH STRENGTH~~ > 13,708 lb/ft

THIS IS STIFFNESS NOT STRENGTH!

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Tensar®

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Product Specification - Biaxial Geogrid BX1100

Tensar International Corporation reserves the right to change its product specifications at any time. It is the responsibility of the specifier and purchaser to ensure that product specifications used for design and procurement purposes are current and consistent with the products used in each instance.

Product Type:	**Integrally Formed Biaxial Geogrid**
Polymer:	**Polypropylene**
Load Transfer Mechanism:	**Positive Mechanical Interlock**
Primary Applications:	**Spectra System (Base Reinforcement, Subgrade Improvement)**

Product Properties

Index Properties	**Units**	**MD Values[1]**	**XMD Values[1]**
▪ Aperture Dimensions[2]	mm (in)	25 (1.0)	33 (1.3)
▪ Minimum Rib Thickness[2]	mm (in)	0.76 (0.03)	0.76 (0.03)
▪ Tensile Strength @ 2% Strain[3]	kN/m (lb/ft)	4.1 (280)	6.6 (450)
▪ Tensile Strength @ 5% Strain[3]	kN/m (lb/ft)	8.5 (580)	13.4 (920)
▪ Ultimate Tensile Strength[3]	kN/m (lb/ft)	12.4 (850)	19.0 (1,300)
Structural Integrity			
▪ Junction Efficiency[4]	%	93	
▪ Flexural Stiffness[5]	mg-cm	250,000	
▪ Aperture Stability[6]	m-N/deg	0.32	
Durability			
▪ Resistance to Installation Damage[7]	%SC / %SW / %GP	95 / 93 / 90	
▪ Resistance to Long Term Degradation[8]	%	100	
▪ Resistance to UV Degradation[9]	%	100	

Dimensions and Delivery

The biaxial geogrid shall be delivered to the jobsite in roll form with each roll individually identified and nominally measuring 3.0 meters (9.8 feet) or 4.0 meters (13.1 feet) in width and 75.0 meters (246 feet) in length. A typical truckload quantity is 185 to 250 rolls.

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02. Brief descriptions of test procedures are given in the following notes.
2. Nominal dimensions.
3. Determined in accordance with ASTM D6637-10 Method A.
4. Load transfer capability determined in accordance with ASTM D7737-11.
5. Resistance to bending force determined in accordance with ASTM D7748-12, using specimens of width two ribs wide, with transverse ribs cut flush with exterior edges of longitudinal ribs, and of length sufficiently long to enable measurement of the overhang dimension.
6. Resistance to in-plane rotational movement measured by applying a 20 kg-cm (2 m-N) moment to the central junction of a 9 inch x 9 inch specimen restrained at its perimeter in accordance with GRI GG9.
7. Resistance to loss of load capacity or structural integrity when subjected to mechanical installation stress in clayey sand (SC), well graded sand (SW), and crushed stone classified as poorly graded gravel (GP). The geogrid shall be sampled in accordance with ASTM D5818 and load capacity shall be determined in accordance with ASTM D6637.
8. Resistance to loss of load capacity or structural integrity when subjected to chemically aggressive environments in accordance with EPA 9090 immersion testing.
9. Resistance to loss of load capacity or structural integrity when subjected to 500 hours of ultraviolet light and aggressive weathering in accordance with ASTM D4355-05.

This product specification supersedes all prior specifications for the product described above and is not applicable to any products shipped prior to February 1, 2013.

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Tensar®

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2500 Northwinds Pkwy, Suite 500
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Product Specification - Biaxial Geogrid BX1500

Tensar International Corporation reserves the right to change its product specifications at any time. It is the responsibility of the specifier and purchaser to ensure that product specifications used for design and procurement purposes are current and consistent with the products used in each instance.

Product Type:	**Integrally Formed Biaxial Geogrid**
Polymer:	**Polypropylene**
Load Transfer Mechanism:	**Positive Mechanical Interlock**
Primary Applications:	**Spectra System (Base Reinforcement, Subgrade Improvement)**

Product Properties

Index Properties	**Units**	**MD Values[1]**	**XMD Values[1]**
▪ Aperture Dimensions[2]	mm (in)	25 (1.0)	30.5 (1.2)
▪ Minimum Rib Thickness[2]	mm (in)	1.78 (0.07)	1.78 (0.07)
▪ Tensile Strength @ 2% Strain[3]	kN/m (lb/ft)	8.5 (580)	10.0 (690)
▪ Tensile Strength @ 5% Strain[3]	kN/m (lb/ft)	17.5 (1,200)	20.0 (1,370)
▪ Ultimate Tensile Strength[3]	kN/m (lb/ft)	27.0 (1,850)	30.0 (2,050)
Structural Integrity			
▪ Junction Efficiency[4]	%	93	
▪ Flexural Stiffness[5]	mg-cm	2,000,000	
▪ Aperture Stability[6]	m-N/deg	0.75	
Durability			
▪ Resistance to Installation Damage[7]	%SC / %SW / %GP	95 / 93 / 90	
▪ Resistance to Long Term Degradation[8]	%	100	
▪ Resistance to UV Degradation[9]	%	100	

Dimensions and Delivery

The biaxial geogrid shall be delivered to the jobsite in roll form with each roll individually identified and nominally measuring 4.0 meters (13.1 feet) in width and 50.0 meters (164 feet) in length. A typical truckload quantity is 180 rolls.

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02. Brief descriptions of test procedures are given in the following notes.
2. Nominal dimensions.
3. Determined in accordance with ASTM D6637-10 Method A.
4. Load transfer capability determined in accordance with ASTM D7737-11.
5. Resistance to bending force determined in accordance with ASTM D7748-12, using specimens of width two ribs wide, with transverse ribs cut flush with exterior edges of longitudinal ribs, and of length sufficiently long to enable measurement of the overhang dimension.
6. Resistance to in-plane rotational movement measured by applying a 20 kg-cm (2 m-N) moment to the central junction of a 9 inch x 9 inch specimen restrained at its perimeter in accordance with GRI GG9.
7. Resistance to loss of load capacity or structural integrity when subjected to mechanical installation stress in clayey sand (SC), well graded sand (SW), and crushed stone classified as poorly graded gravel (GP). The geogrid shall be sampled in accordance with ASTM D5818 and load capacity shall be determined in accordance with ASTM D6637.
8. Resistance to loss of load capacity or structural integrity when subjected to chemically aggressive environments in accordance with EPA 9090 immersion testing.
9. Resistance to loss of load capacity or structural integrity when subjected to 500 hours of ultraviolet light and aggressive weathering in accordance with ASTM D4355-05.

Tensar International Corporation warrants that at the time of delivery the geogrid furnished hereunder shall conform to the specification stated herein. Any other warranty including merchantability and fitness for a particular purpose, are hereby excluded. If the geogrid does not meet the specifications on this page and Tensar is notified prior to installation, Tensar will replace the geogrid at no cost to the customer.

This product specification supersedes all prior specifications for the product described above and is not applicable to any products shipped prior to February 1, 2013.

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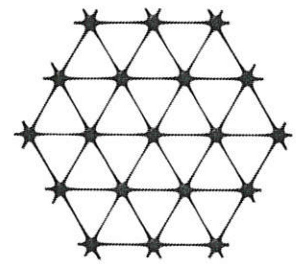
Tensar. THE COMPANY YOU CAN BUILD ON®

Product Specification - TriAx® TX130S Geogrid[1]

Tensar TriAx® Geogrid



General

1. The geogrid is manufactured from a punched polypropylene sheet, which is then oriented in three substantially equilateral directions so that the resulting ribs shall have a high degree of molecular orientation, which continues at least in part through the mass of the integral node.
2. The properties contributing to the performance of a mechanically stabilized layer include the following:

Index Properties	Longitudinal	Diagonal	General
▪ Rib pitch[2], mm (in)	33 (1.30)	33 (1.30)	
▪ Rib shape			Rectangular
▪ Aperture shape			Triangular

Structural Integrity	
▪ Junction efficiency[3], %	93
▪ Overall Flexural Rigidity[4], mg-cm	500,000
▪ Radial stiffness at low strain[5], kN/m @ 0.5% strain	200
(lb/ft @ 0.5% strain)	(13,708)

Durability	
▪ Resistance to chemical degradation[6]	100%
▪ Resistance to ultra-violet light and weathering[7]	70%

Dimensions and Delivery

The TX geogrid shall be delivered to the jobsite in roll form with each roll individually identified and nominally measuring 3.0 meters (9.8 feet) and/or 4.0 meters (13.1feet) in width and 75 meters (246 feet) in length.

Notes

1. Unless indicated otherwise, values shown are minimum average roll values determined in accordance with ASTM D4759-02. Brief descriptions of test procedures are given in the following notes.
2. Nominal dimensions.
3. Load transfer capability determined in accordance with ASTM D6637-10 and ASTM D7737-11 and expressed as a percentage of ultimate tensile strength.
4. Determined in accordance with ASTM D7748-12.
5. Radial stiffness is determined from tensile stiffness measured in any in-plane axis from testing in accordance with ASTM D6637-10.
6. Resistance to loss of load capacity or structural integrity when subjected to chemically aggressive environments in accordance with EPA 9090 immersion testing.
7. Resistance to loss of load capacity or structural integrity when subjected to 500 hours of ultraviolet light and aggressive weathering in accordance with ASTM D4355-05.

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RK&K
81 Mosher Street
Baltimore, Maryland 21217

GEOTECHNICAL QUALITY CONTROL SIGN OFF SHEET

Project Name: Christina River Bridge

Commission Number: 04-130-03G

Client Name: DelDOT

Document Title: Lateral Squeeze Analysis of Roadway Emb. STA 447+00 to 450+00

Document Type: (Check One)
- ☐ Proposal
- ☐ Report/Memo
- ☒ Calculations
- ☐ Boring Logs
- ☐ Specification
- ☐ Other: ____

Project Milestone: ____ **Due Date:** ____

This is to certify that I was the originator of this work product, and that I have carefully checked my work prior to passing it on for Quality Control checking.

Printed Name: Bibek Shrestha

Signature: Bibek Shrestha **Date:** 3/10/2016

	Print Name	Signature	Date
☑ Checked:	ERIC M KLEIN	E M Klein	4/26/16
☐ Corrected:			
☐ Checked:			
☐ Corrected:			
☐ Checked:			
☐ Project Engr:			
☑ PM/Manager:	ERIC KLEIN	E M Klein	4/26/16

RK&K
81 Mosher Street
Baltimore, Maryland 21217

GEOTECHNICAL CALCULATION QUALITY CONTROL CHECKLIST

Project: Christina River Bridge **Comm. No:** 04-130-03G

Title of Calculation Set: Lateral Squeeze Analysis of Roadway Emb. STA 447+00 to 450+00

RK&K Project Manager: EMK **Coordinating Discipline POC:** ______

Calculations Prepared By: BBS **Design Method/Code:** ______

File Path on Network: ______

Subsurface Characterization Approved By: ______

Initial Calculations: Signature of Originator: Bibek Shrestha Date: 3/10/2016

- ☒ Geotech Quality Control Sign Off Sheet is provided.
- ☒ Calculations are neat, legible and understandable.
- ☒ Variable inputs are highlighted and check confirmations are clearly marked in calculation spreadsheets.
- ☐ Table of Contents is provided.
- ☒ Heading on each page filled out, including computer input and output.
- ☒ The first page contains purpose of the calculation.
- ☒ References for design criteria, data, methods, formulas and computer program names and version numbers are given, and included in appendices as necessary.
- ☒ All assumptions are listed, explained, and those needed to be verified later are flagged.
- ☐ All input by others is attached and includes originator, date provided. (i.e. Structures loads, concrete/steel properties, etc.)
- ☒ Can the analytical steps involved be followed independent of the originator?
- ☒ Do the calculations agree with the other inter-department / project documents?
- ☐ Footer with file path/tab name and print date is provided on calculation spreadsheets.
- ☐ Detailed CAD ready sketches are provided that clearly depict the design requirements.
- ☒ Calculation Summary with recommendations included in front of calculation set.
- ☒ All calculations are checked and each calculation sheet is initialed by the originator before giving to the checker.

Finalized Calculations: Signature of Originator: ______ Date: ______

- ☐ All pages are numbered sequentially and cross-references are complete and accurate.
- ☐ All calculations are checked and each calculation sheet initialed by the checker.
- ☐ Calculation set reviewed by Project Manager.
- ☐ Has this calculation been superseded? Reference new calculation, if applicable.

This sheet is to be prepared by the originator of the calculation and attached to the front of the calculation set

PURPOSE: EVALUATE THE POTENTIAL FOR LATERAL SQUEEZE OF STRATUM Ia DUE TO THE CONSTRUCTION OF ROADWAY EMBANKMENT

REFERENCE: CHECK LATERAL SQUEEZE FOR EMBANKMENTS BASED ON SILVESTRI 1983

$$FS_{SQUEEZING} = \frac{2c_u}{\Gamma D_s \tan\theta} + \frac{4.14\,c_u}{H\Gamma} \geq 1.3$$

SEE ATTACHED SHEETS.

θ = ANGLE OF SLOPE

Γ = UNIT WEIGHT OF SOIL IN SLOPE

D_s = DEPTH OF SOIL BENEATH SLOPE BASE OF EMBANKMENT

H = HEIGHT OF SLOPE

c_u = UNDRAINED SHEAR STRENGTH OF SOFT SOIL BENEATH SLOPE

\# SEE CALCSET ROADWAY EMBANKMENT STA 447+00 TO STA 450+00 DATED 7/15/15 BY BBS

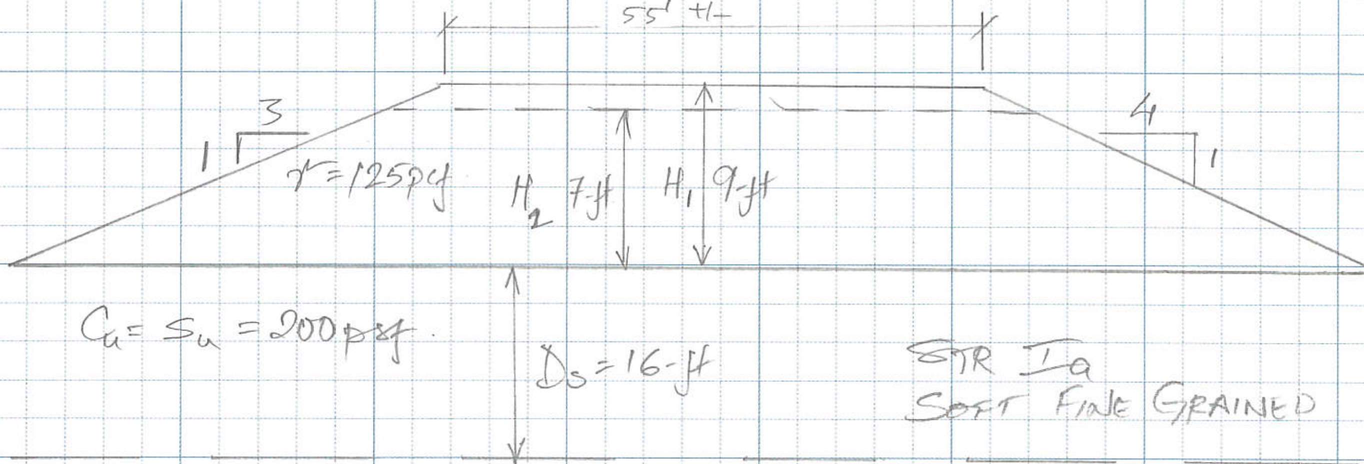
ASSUMPTIONS:

1) THICKNESS OF SOFT LAYER (STR Ia) = 16-ft (AVG)

2) EMBANKMENT SLOPE 3H:1V

3) EMBANKMENT TEMP. HEIGHT 9-ft FOR SURCHARGE & 7-ft FOR FINAL CONDITION

SOIL PARAMETE & CROSS SECTION



NTS

FOR 3H:1V SLOPE $\theta = 18.44°$

$$FS_{SQUEEZING} = \frac{2C_u}{\gamma D_s \tan\phi} + \frac{4.14\,C_u}{H\gamma} > 1.3$$

TEMPORARY COND^N FOR H = 9-ft

$$FS_s = \frac{2 \times 200}{125 \times 16 \times \tan 18.44} + \frac{4.14 \times 200}{9 \times 125}$$

$$= 0.6 + 0.736$$

$$FS = 1.336 > 1.3 \text{ OK}$$

FOR PERMANENT CONDITION H = 7-ft

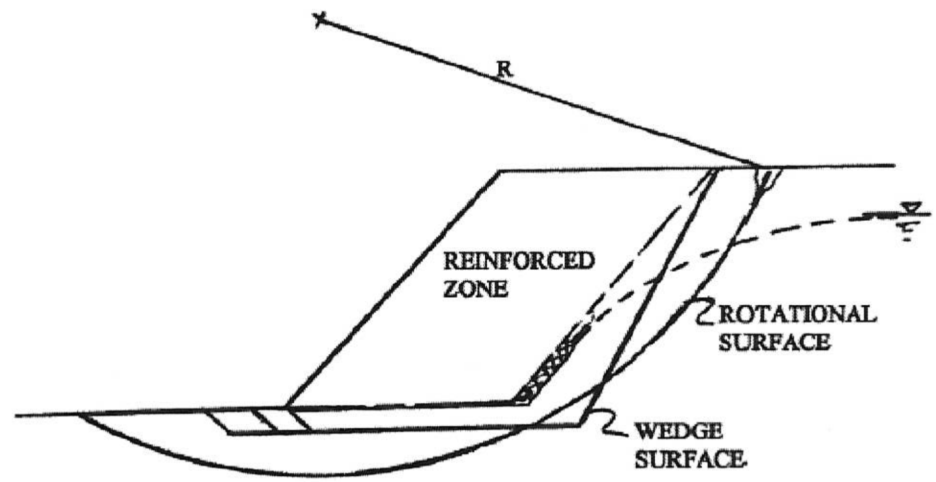
$$FS_s = \frac{2 \times 200}{125 \times 16 \times \tan 18.44} + \frac{4.14 \times 200}{7 \times 125}$$

$$= 0.6 + 0.946$$

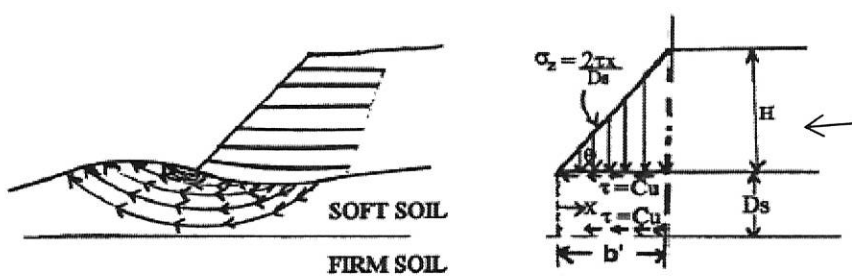
$$FS_s = 1.546 \text{ OK}$$

NOTE AFTER PRELOADING & SHEAR STRENGTH GAIN IN STR Ia FACTOR OF SAFETY WILL INCREASE.

Lateral Squeeze



a) Deep seated (global) stability analysis.



$$FS = \frac{2\,c_u}{\gamma\,D_s \tan\theta} + \frac{4.14\,c_u}{H\,\gamma}$$

b) Local bearing failure (lateral squeeze)

Figure 9-8. Failure through the foundation.

Interesting Model of Squeeze

Lateral Squeeze

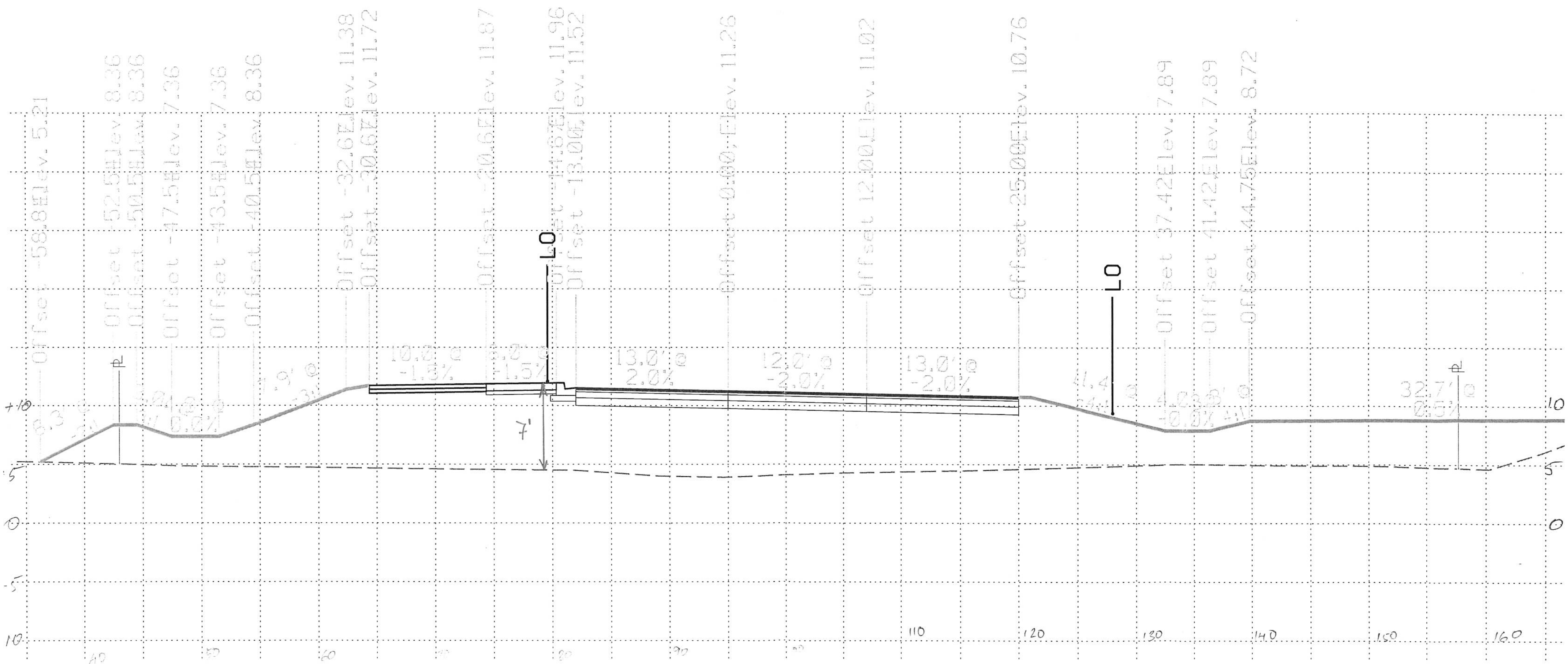
- Local bearing failure at the toe (lateral squeeze) (Figure 9-8b).
  - If a weak soil layer exists beneath the embankment to a limited depth D_S which is less than the width of the slope b', the factor of safety against failure by squeezing may be calculated from (Silvestri, 1983):

$$FS_{squeezing} = \frac{2c_u}{\gamma D_s \tan\theta} + \frac{4.14c_u}{H\gamma} \geq 1.3 \qquad (9\text{-}15)$$

where:

θ = angle of slope
γ = unit weight of soil in slope
D_s = depth of soft soil beneath slope base of the embankment
H = height of slope
c_u = undrained shear strength of soft soil beneath slope

Caution is advised and rigorous analysis (e.g., numerical modeling) should be performed when FS < 2. This approach is somewhat conservative as it does not provide any influence from the reinforcement. When the depth of the soft layer, D_S, is greater than the slope base width, b', general slope stability will govern design.


Offset -58.8 Elev. 5.21
Offset -52.5 Elev. 8.36
Offset -50.5 Elev. 8.36
Offset -47.5 Elev. 7.36
Offset -43.5 Elev. 7.36
Offset -40.5 Elev. 8.36
Offset -32.6 Elev. 11.38
Offset -30.6 Elev. 11.72
Offset -20.6 Elev. 11.87
LO
Offset -14.87 Elev. 11.96
Offset -13.00 Elev. 11.52
Offset 0.00 Elev. 11.26
Offset 12.00 Elev. 11.02
Offset 25.00 Elev. 10.76
LO
Offset 37.42 Elev. 7.89
Offset 41.42 Elev. 7.89
Offset 44.75 Elev. 8.72
10.0' @ -1.5%
6.0' @ -1.5%
13.0' @ 2.0%
12.0' @ -2.0%
13.0' @ -2.0%
7'
32.7' @ 0.5%
+10
5
0
-5
-10
10
5
0
40
50
60
110
120
130
140
150
160

STA 448+50

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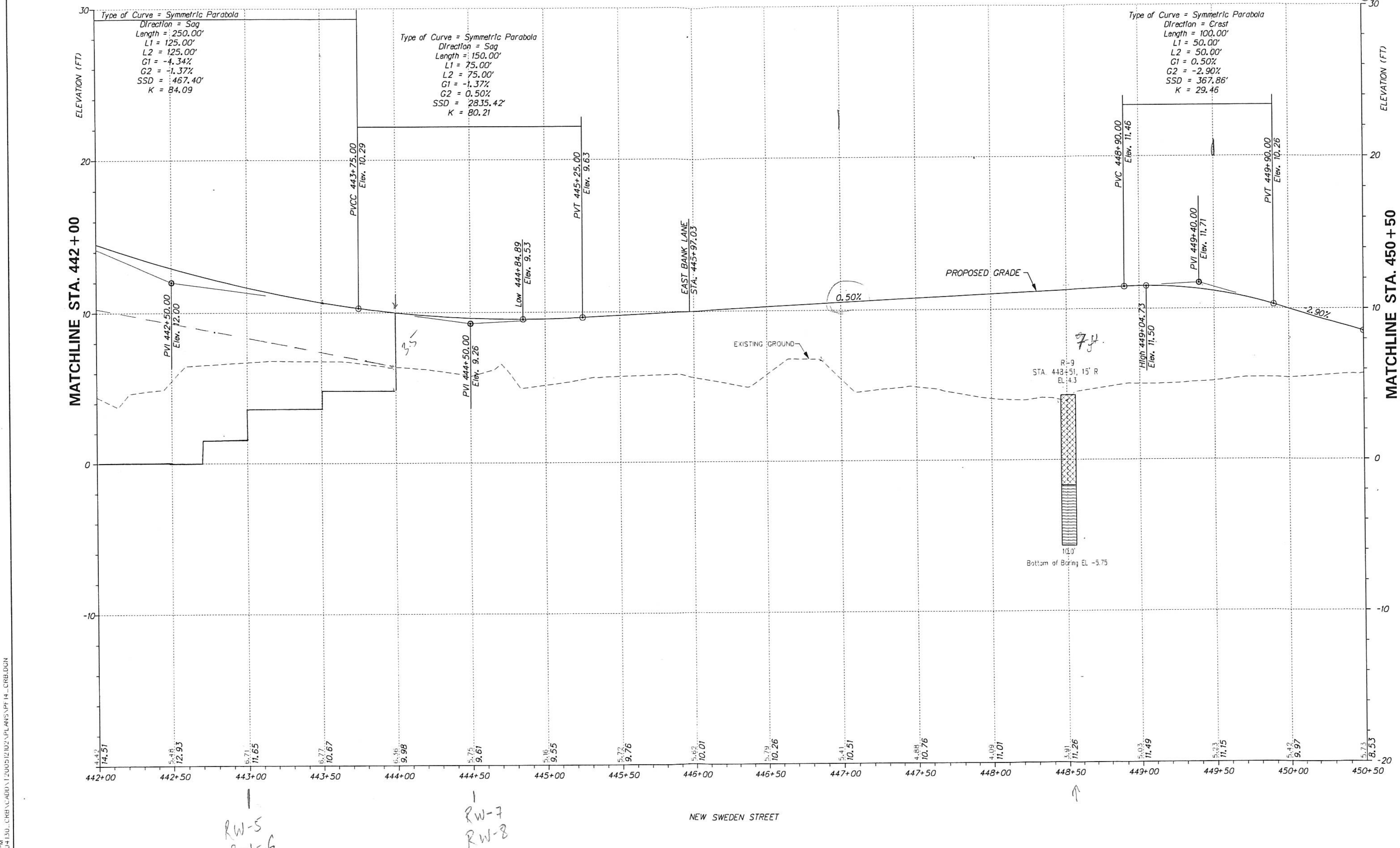

DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	HORIZONTAL SCALE 0 30 60 90 FEET	CHRISTINA RIVER BRIDGE AND APPROACHES	CONTRACT T200512102 / COUNTY NEW CASTLE / BRIDGE NO. 1-691 / DESIGNED BY: MK / CHECKED BY: DS	PROFILES

SHEET NO. 65